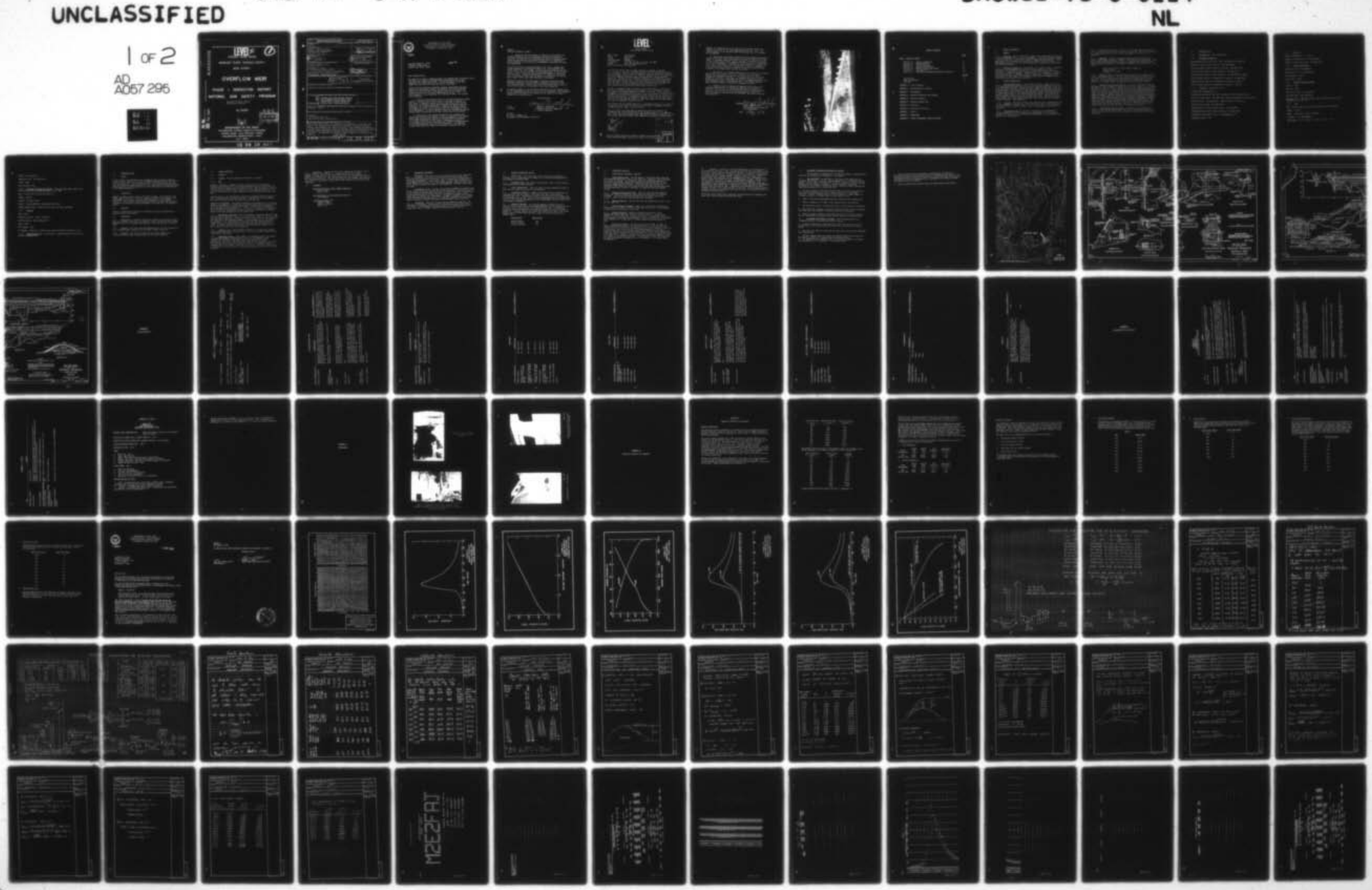
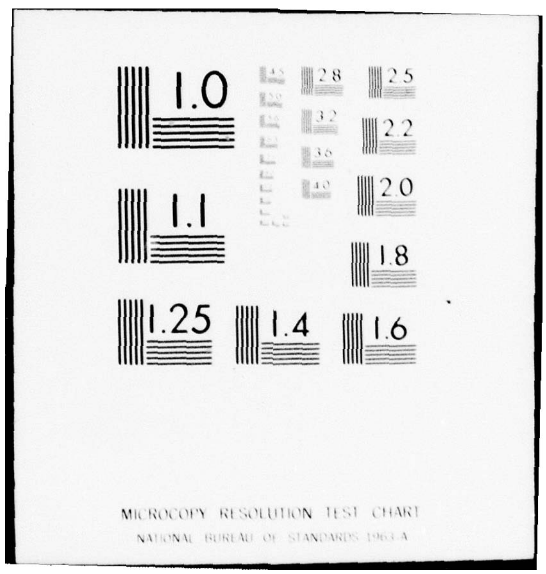
AD-A057 295 GILBERT ASSOCIATES INC READING PA F/G 13/2
NATIONAL DAM SAFETY PROGRAM. OVERFLOW WEIR (NJ00214), PASSAIC R--ETC(U)
JUL 78 J A HAGEN DACW61-78-C-0114

UNCLASSIFIED NL

1 OF 2
AD
A057 295




1.0
1.1
1.25
1.4
1.6
1.8
2.0
2.2
2.5
2.8
3.2
3.6
4.0
MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS 1963-A

PASSAIC RIVER BASIN

WANAQUE RIVER, PASSAIC COUNTY

NEW JERSEY

OVERFLOW WEIR

PHASE I INSPECTION REPORT

NATIONAL DAM SAFETY PROGRAM

NJ 00214

DEPARTMENT OF THE ARMY
PHILADELPHIA DISTRICT, CORPS OF ENGINEERS
CUSTOM HOUSE - 2D & CHESTNUT STREETS
PHILADELPHIA, PENNSYLVANIA 19106
JULY 1978

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER NJ00214	2. GOVT ACCESSION NO.	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) Phase I Inspection Report National Dam Safety Program Overflow Weir (NJ00214) Passaic County, N.J.		5. TYPE OF REPORT & PERIOD COVERED 9 FINAL rept. 6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) 10 James A. Hagen, P.E.		8. CONTRACT OR GRANT NUMBER(s) 15 DACW61-78-C-0114
9. PERFORMING ORGANIZATION NAME AND ADDRESS Gilbert Associates, Inc. P.O. Box 1489 Reading, Pa. 19603		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS U.S. Army Engineer District, Philadelphia Custom House, 2d & Chestnut Streets Philadelphia, Pennsylvania 19106		12. REPORT DATE 11 July 1978 13. NUMBER OF PAGES 128
14. MONITORING AGENCY NAME & ADDRESS(if different from Controlling Office) 12 136 p.		15. SECURITY CLASS. (of this report) Unclassified 15a. DECLASSIFICATION/DOWNGRADING SCHEDULE
16. DISTRIBUTION STATEMENT (of this Report) Approved for public release; distribution unlimited.		
17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report) 6 National Dam Safety Program. Overflow Weir (NJ00214), Passaic River Basin, Wanaque River, Passaic County, New Jersey. Phase I Inspection Report.		
18. SUPPLEMENTARY NOTES Copies are obtainable from National Technical Information Service, Springfield, Virginia, 22151.		
19. KEY WORDS (Continue on reverse side if necessary and identify by block number) Dams--N.J. Overflow Weir Dam, N.J. National Dam Safety Program Phase I		
20. ABSTRACT (Continue on reverse side if necessary and identify by block number) This report cites results of a technical investigation as to the dam's adequacy. The inspection and evaluation of the dam is as prescribed by the National Dam Inspection Act, Public Law 92-367. The technical investigation includes visual inspection, review of available design and construction records, and preliminary structural and hydraulic and hydrologic calculations, as applicable. An assessment of the dam's general condition is included in the report. 154 850		

DD FORM 1 JAN 73 1473 EDITION OF 1 NOV 65 IS OBSOLETE

78 08 10 005

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

DEPARTMENT OF THE ARMY
PHILADELPHIA DISTRICT, CORPS OF ENGINEERS
CUSTOM HOUSE—2 D & CHESTNUT STREETS
PHILADELPHIA, PENNSYLVANIA 19106

IN REPLY REFER TO
NAPEN-D

8 AUG 1978

Honorable Brendan T. Byrne
Governor of New Jersey
Trenton, New Jersey 08621

Dear Governor Byrne:

Inclosed is the Phase I Inspection Report for Overflow Weir in Passaic County, New Jersey which has been prepared under authorization of the Dam Inspection Act, Public Law 92-367. A brief assessment of the dam's condition is given on the first two pages of the report.

Based on visual inspection, available records, calculations and past operational performance, *Overflow is judged to be in fair condition.* However, the overflow weir is not able to pass the PMF without overtopping the portion of the earth embankments below elev. 308.8. To insure adequacy of the structure, the following actions, as a minimum, are recommended:

a. Hydrologic and hydraulic investigations and engineering studies should be initiated within three months of the date of approval of this report to determine corrective action required to increase the capacity of the spillway and/or obtain adequate freeboard to prevent overtopping of the earth embankments. Construction should commence in calendar year 1979. Due to the potential for overtopping of the earth embankments, a detailed emergency operation, drawdown and warning system should be developed by the owner within the next two months.

b. Monitoring of the seepage through the left retaining wall should begin within thirty days of the date of approval of this report. Engineering investigations should be initiated within four months of the date of approval of this report to determine the source of this seepage and to further investigate and perform a seepage analysis of the earth embankment south of the overflow weir. Any remedial measures found necessary should be initiated in calendar year 1979.

ata Entered)

NAPEN-D
Honorable Brendan T. Byrne

c. Within one year of the date of approval of this report the scouring problem at the left downstream retaining wall should be corrected, the scaling of the retaining wall and the footbridge piers should be corrected and the trees and brush should be removed the earth embankment to the south and the embankment stabilized with suitable vegetation.

A copy of the report is being furnished to Mr. Dirk C. Hofman, New Jersey Department of Environmental Protection, the designated State Office contact for this program. Within five days of the date of this letter, a copy will also be sent to Congressman Robert A. Roe of the Eighth District. Under the provisions of the Freedom of Information Act, the inspection report will be subject to release by this office, upon request, thirty days after the date of this letter.

Additional copies of this report may be obtained from the National Technical Information Services (NTIS), Springfield, Virginia, 22161 at a reasonable cost. Please allow four to six weeks from the date of this letter for NTIS to have copies of the report available.

An important aspect of the Dam Safety Program will be the implementation of the recommendations made as a result of the inspection. We accordingly request that we be advised of proposed actions taken by the State to implement our recommendations.

Sincerely yours,

Harry V. Dutchyshyn

HARRY V. DUTCHYSHYN
Colonel, Corps of Engineers
District Engineer

1 Incl
As stated

Cy furn:
Mr. Dirk C. Hofman, P.E.
Department of Environmental Protection

LEVEL II

Phase I Report
National Dam Safety Program

Name of Dam: Overflow Weir
State: New Jersey
County: Passaic
USGS Quad Sheet: Wanaque, N. J.
Coordinates: N 41° 02' 33" LAT., W 74° 17' 50" LONG.
Stream: None (Off the Wanaque River)
Dates of Inspection: 9-10 May 1978

This concrete gravity dam is in fair condition as defined in Appendix J. It is one of nine dams on Wanaque Reservoir and the only design outlet for flood discharge. There is some scouring under the left downstream retaining wall which should be repaired soon. There is seepage with one estimated flow of two to five gallons per minute coming through this wall under high water conditions. An area of severe concrete scaling is evident below the cracked gunite surfacing on this wall as well as on several of the footbridge piers above the weir. The wall seepage should be closely monitored starting very soon, the source of the leak should be determined soon, and areas of deteriorated concrete should be rehabilitated in the future.

The Weir is inadequate under screening criteria established by the Corps for this project because the earth fills adjacent to the Overflow Weir will be overtopped under Probable Maximum Flood (PMF) conditions. Future studies are recommended to determine what the design flood of the reservoir should be and to determine additional flood discharge or storage capacity for the reservoir as indicated by the study. A conventional margin of safety against overturning does not exist under design loading for this dam, therefore a seismic stability study must be performed in the near future.

The drawdown time for Wanaque Reservoir is considered excessive, so a means for reducing this time should be studied soon and a means to implement the study recommendations provided in the near future.

There is a short earth dam south of the main structure for which it is recommended that further investigation and seepage analysis to be conducted soon. Also, the trees growing on this dam should be removed in the near future and then the resultant cleared slopes immediately provided with grass cover.

JAMES A. HAGEN
REGISTERED PROFESSIONAL ENGINEER
DELAWARE

Based on visual inspection, available records, calculations and past operational performance, Overflow is judged to be in fair condition.

However, the overflow weir is not able to pass the PMF without overtopping the portion of the earth embankments below elev. 308.8. To insure adequacy of the structure, the following actions, as a minimum, are recommended:

a. Hydrologic and hydraulic investigations and engineering studies should be initiated within three months of the date of approval of this report to determine corrective action required to increase the capacity of the spillway and/or obtain adequate freeboard to prevent overtopping of the earth embankments. Construction should commence in calendar year 1979. Due to the potential for overtopping of the earth embankments, a detailed emergency operation, drawdown and warning system should be developed by the owner within the next two months.

b. Monitoring of the seepage through the left retaining wall should begin within thirty days of the date of approval of this report. Engineering investigations should be initiated within four months of the date of approval of this report to determine the source of this seepage and to further investigate and perform a seepage analysis of the earth embankment south of the overflow weir. Any remedial measures found necessary should be initiated in calendar year 1979.

c. Within one year of the date of approval of this report the scouring problem at the left downstream retaining wall should be corrected, the scaling of the retaining wall and the footbridge piers should be corrected and the trees and brush should be removed the earth embankment to the south and the embankment stabilized with suitable vegetation.

APPROVED: Harry V. Dutchyshyn
HARRY V. DUTCHYSHYN
Colonel, Corps of Engineers
District Engineer

DATE: 3 Aug 1978

May 1978

OVERFLOW WEIR

TABLE OF CONTENTS

1.0 PROJECT INFORMATION

1.1 GENERAL

1.1.1 Authority: Public Law 92-367, 8 August 1972, authorized the Secretary of the Army, through the U.S. Corps of Engineers to initiate a national program of safety inspections of non-Federal dams throughout the United States. Gilbert Associates, Inc. has entered into Contract No. DACW61-78-C-0114 with the Philadelphia Office of the U.S. Corps of Engineers to inspect this dam, Gilbert Work Order 06-7249-000.

1.1.2 Purpose of Inspection: The purpose is to conduct a Phase I inspection according to the Recommended Guidelines for Safety Inspection of Dams (Reference 1 of Appendix G) and the terms of the Gilbert Associates contract with the U.S. Corps of Engineers. The objectives are to expeditiously identify those dams which pose an immediate threat to human life or property, gather information for this report, and recommend future studies or remedial actions where they are indicated by the inspection.

1.2 PROJECT DESCRIPTION

1.2.1 Dam and Appurtenances: The Overflow Weir dam is composed of a 552 foot long mass concrete gravity structure with a maximum height of 30 feet and is an ogee-shaped spillway. Thirty-nine piers support a footbridge on the top of the spillway. The dam currently has three timber stoplogs above the spillway which was earlier raised about 0.2 feet when the spillway surface was gunited for maintenance reasons. The downstream features include a stilling basin, two concrete channel dams and an overflow downstream channel. The spillway was built diagonally across a series of foliated rock strata. There is a 17 foot high concrete core, earth dike about 50 feet long located 100 feet southwest of the main structure. (See Appendix I).

1.2.2 Location: The dam is located about 300 feet south of Raymond Dam and 1/2 mile northwest of Wanaque, New Jersey in Passaic County. The location of the Overflow Weir is shown on Figure 1 and on the geological map attached as Appendix F.

1.2.3 Size Classification: The dam is classified as an intermediate structure because of its 30' height and its impoundment (44,350 acre-feet between crest elevation 302.4 and bottom elevation 280.0), in accordance with Section 2.1.1 of Reference 1.

1.2.4 Hazard Classification: The dam is located upstream of a populated valley and floodplain area, which includes several towns. The dam is classified as a high hazard potential based on the requirements of Section 2.1.2 of Reference 1.

1.2.5 Ownership: The dam is owned and maintained by the North Jersey District Water Supply Commission (NJDWSC), a New Jersey state commission. They have engineering and maintenance facilities at Raymond Dam in Wanaque, New Jersey. The Chief Engineer of the NJDWSC in Wanaque is Mr. Dean C. Noll. The address is:

North Jersey District Water Supply Commission
Ringwood Avenue
Wanaque, N. J. 07465

1.2.6 Purpose of Dam: The dam was designed to pass the original design flood of the Wanaque Project. In 1934, three flashboards were added to raise the elevation by 23 inches and provide additional storage capacity in the reservoir. The Wanaque Reservoir supplies water to residents of Paterson, Montclair, Glen Ridge, Newark, Kearny, Passaic and Clifton, New Jersey.

1.2.7 Design and Construction History: The dam was constructed between September 10, 1926 and June 25, 1928 by the Clifford F. MacEvoy Co. of Newark, New Jersey as part of the total Wanaque Project. The project began with the construction contract for the lower portion of Wanaque (Raymond) Dam which was awarded in 1920 and was completed with the reservoir being filled by March 4, 1929. The original design records could not be located by the staff of the NJDWSC at Wanaque. However, publications indicate the design was performed by employees of the NJWDSC with assistance of individual consultants. The New Jersey Department of Environmental Protection (DEP) has some monthly progress inspection reports and several photographs taken during construction. The addition of the permanent stoplogs in 1934 and the guniting of the surface around 1966 (by A. Belanger and Sons Contract No. 149) were the only major revisions and repairs to the dam since its original construction.

1.2.8 Normal Operational Procedures: The dam relies on uncontrolled overflow across the spillway to pass storm flow from the Wanaque Reservoir. There is no operational procedure for removing the flashboards during flood conditions.

1.3 PERTINENT DATA

1.3.1 Drainage Area: 90.4 square miles

1.3.2 Discharge at Dam Site:

Maximum known flood at dam site: 8470 cfs; Reservoir El.303.9 ft.

Warm water outlet at pool elevation: Not applicable.

Diversion tunnel low pool outlet at elevation: Not Applicable.

Diversion tunnel outlet at pool elevation: Not Applicable.

Gated spillway capacity at pool elevation: Not Applicable (no gates)

Gated spillway capacity at maximum pool elevation: Not Applicable.

Ungated spillway capacity at maximum pool elevation: 27900 cfs.

Total spillway capacity at maximum pool elevation: 27900 cfs.

1.3.3 Elevation: (Feet above M.S.L.)

Top Dam with flashboards 302.4

Maximum Spillway Design Flood (SDF) surcharge: 308.8 (See Section 5.0).

Full flood control pool: Not Applicable. (Dam not regulated for flood control).

Recreation pool: Not Applicable. (Dam not used for recreation).

Spillway crest (gated): Not Applicable.

Upstream portal invert diversion tunnel: Not Applicable.

Downstream portal invert diversion tunnel: Not Applicable.

Streambed at centerline of dam: 272 (Topographic low)

Maximum tailwater: 290

1.3.4 Reservoir

Length of maximum pool: 6.6 miles

Length of recreation pool: Not Applicable.

Length of flood control pool: Not Applicable.

1.3.5 Storage (Acre-feet)

Recreation Pool: Not Applicable.

Flood Control Pool: Not Applicable.

SDF Surcharge: 58,309

Top of Dam: 44,350

1.3.6 Reservoir Surface (Acres)

Top Dam: 2400

SDF pool: 2590

Flood control pool: Not Applicable.

Recreation pool: Not Applicable.

Spillway crest: 2400 (top of dam is top of spillway and also top of flashboards - El. 302.4 ft)

1.3.7 Dam

Type: Concrete gravity dam with ogee spillway and flashboards

Length: 552 feet

Height: 30 feet max. (15 ft ± average)

Top Width: N.A. (Ogee Shaped)

Side Slopes: U/S - 1.5 (H): 1 (V), D/S - 0.6 (H): 1 (V)

Zoning: Not Applicable.

Impervious Core: Not Applicable.

Cutoff: None

Grout curtain: None

1.3.8 Diversion and Regulating Tunnel: (Low pool drain pipe; there is no true diversion and/or regulating tunnel at this dam).

Type: 4-inch cast iron pipe (See Figure 2)

Length: 25 feet

Closure: Two Gate valves

Access: 24-inch manhole on downstream face of dam

Regulating Facilities: Manual (used to drain low pocket upstream)

1.3.9 Spillway

Type: Ogee

Length of Spillway: 520 ft (effective)

Crest Elevation: With stoplogs 302.4

Gates: None

U/S, Channel: None

D/S Channel: Very good - exposed rock, deeply entrenched in gneissic rocks

1.3.10 Regulating Outlets: Not Available. (Wanaque Reservoir can be slowly drained at Raymond Dam)

2.0 ENGINEERING DATA

2.1 DESIGN

A plan, profile, and maximum section through the dam are shown on original record tracings which are on file at the NJDWSC engineering office (Mr. Dean C. Noll) at Wanaque, N.J. (See attached Figures). No original design data were available other than results mentioned in the North East Water Works Association publication "Public Works" (Reference 2) and a 1925 report by the Commissioner of the NJDWSC (Reference 3).

2.2 CONSTRUCTION

Contract drawings, specifications, and record drawings, are available at the NJDWSC engineering office. Periodic inspection reports, news clippings, and photographs are available at the New Jersey Department of Environmental Protection. They indicate that satisfactory work was performed on the project in general and indicated no unusual problems at this dam site. See Figures 2 and 3.

2.3 OPERATION

There is no operational procedure for removing the spillway flashboards or otherwise controlling overflow.

2.4 EVALUATION

2.4.1 Availability: Foundation exploration, design, and construction data were not available. Structural and hydraulic design calculations were lacking. Reservoir water level readings were available. Also, see Section 2.1 and 2.2 above.

2.4.2 Adequacy: The record drawings supplemented by field data gathered on this inspection appear adequate for this Phase I safety inspection.

2.4.3 Validity: The record drawings and water levels appear to be consistent with existing structures based on the visual inspection.

3.0 VISUAL INSPECTION

3.1 FINDINGS

3.1.1 General: The area seems well suited for a spillway.

3.1.2 Dam

Seepage or leakage: A seepage of about two gpm (0.004 cfs) through the spalling, left abutment wall, about eight feet beyond the toe of the Weir was observed when the reservoir was at elevation 302.4. This flow was not observed earlier when the reservoir was at elevation 301.5 during the inspection. At reservoir elevation 302.4 slight seepage was viewed coming from the downstream face.

Concrete surface: The structure was resurfaced with mesh reinforced gunite in 1966. Some spalling of the gunited surface is apparent on the left abutment wall and on several piers in the center portion of the dam.

Foundation and abutment: The exposed foundation rocks along the toe of the dam appear to be competent for supporting the superstructure with an adequate factor of safety against bearing failure and sliding. Calculations were performed which confirmed this (See Appendix J). The earth fill behind the abutment walls on each end of the dam showed no evidence of excessive settlement or movement. There is some minor undermining of the abutment wall.

3.1.3 Appurtenant Structures: The stilling basin, apparently shaped by some soil removal, is basically a natural rock lined basin. There are two 20-ft high concrete channel dams; one is located in the stilling basin area, and another at the south end of the downstream channel. These two dams were not included in the inspection list. Between the stilling basin and the downstream overflow channel there is a natural gradient drop within the high-resistant gneissic rocks. In summary, both the stilling basin and the overflow channel were in good condition at the time of this inspection.

3.1.4 Reservoir Area: The Precambrian gneisses are continuously exposed along the shoreline near the Overflow Weir. The various slopes formed by these rocks appear to be stable.

3.1.5 Downstream Channel: This channel is a deepened natural rock channel. A considerable amount of unconsolidated soil cover was removed during construction according to available records and confirmed by visual observations. Portions of the lower channel bottom were stone-paved in a mortar bed which appears to be in good condition. There is no excessive erosion or sedimentation in the channel. The steep rock slopes of the upper channel walls are generally in stable condition. Retaining masonry wall sections are in good condition.

3.2 Evaluation: Judging from the visual inspection, the gravity dam is in good condition (see Appendix J), although several areas of minor seepage or leakage exist at the downstream face and the left abutment wall. Several areas including at least two piers and the left abutment retaining wall were locally deteriorated. The stilling basin, overflow channel, and reservoir area adjacent to the dam were stable and in good condition at the time of the inspection.

3.3 ATTENDEES

North Jersey District Water Supply Commission
Mario Di Laura
Dean C. Noll

New Jersey Dept. of Environmental Protection
Larry Woscyna

Gilbert Associates, Inc.
James A. Hagen
Rudolph J. Wahanik
Fine T. Hsu

4.0 OPERATIONAL PROCEDURES

4.1 PROCEDURES: The flow of water over this spillway is not controlled. Water is retained by the structure to the elevation of the stoplogs which reportedly were permanently installed in 1934 to increase the storage capacity of the Wanaque Reservoir. At all times, water is drawn from the reservoir at nearby Raymond Dam (NJ 00213) for water supply purposes. When the water level in the reservoir exceeds the height of the flashboards, the water flows freely over them.

4.2 MAINTENANCE OF DAM: The dam is maintained by NJDWSC-W personnel based on the results of periodic inspections by their engineers or conditions noted by security personnel during their daily tours of the reservoir rim. The guniting of the concrete surfaces in 1966, periodic replacement of timbers in the footbridge, and the May 1974 replacement of a number of original 1934 stoplogs (which included coating them with a a tar compound, apparently coal tar epoxy) seem to have been the major maintenance items performed on this structure.

4.3 EVALUATION: There are no operational procedures at this dam. The stoplogs are permanently installed and not considered an operational item in this report. The maintenance procedures appear to be generally satisfactory, except that areas of seepage, leakage and undermining were observed.

5.0 HYDRAULIC/HYDROLOGIC DESIGN

5.1 DESIGN DATA: Very little data exists, but some is contained in Reference 3 and in Application File Number 32 of the New Jersey Department of Environmental Protection (DEP).

5.2 EXPERIENCE DATA: The highest recorded water level is 303.93 which is considerably below the PMF elevation.

5.3 VISUAL OBSERVATIONS: There are three stoplogs permanently wedged in place and tar coated on the crest of the spillway.

5.4 OVERTOPPING POTENTIAL: NJDWSC records indicate the flashboards were overtopped less than 50 percent of the years from their installation in 1934-1935 until 1951. The PMF would result in an elevation of flow higher than the footbridge over the dam. Details on the methodology used and the hydrologic results of the spillway performance and the corresponding reservoir water levels are presented in Appendix D.

5.5 RESERVOIR DRAWDOWN: The existing emergency drawdown facilities installed in the several dams of the Wanaque Reservoir are not adequate to lower the water level of the reservoir in a short period of time. A preliminary evaluation of the performance of the existing drawdown facilities is given in Appendix D. The time required to drawdown the Overflow Weir Dam to the bottom surface level of Midvale Dam (280 feet), using the existing facilities at Raymond Dam is:

System in Use	Time in Days
Aerator System	98
36" Dia Blowoff	310
Aerator & Blowoff	75

6.0 STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

6.1.1 Visual Observations: The dam appears to have been stable under the past operating conditions. There is no structural cracking or sign of movement at the base of the dam. The foundation rocks are chiefly gneisses which typically possess high strength and adequate bearing capacity when in fresh state for supporting the structure. A variable final rock surface and a presumably rough excavation rock surface on which the mass concrete was poured, and some downstream rock mass, contribute to foundation stability against sliding.

6.1.2 Design and Construction Data: This data indicates the foundation surface was cleaned by mechanical and/or explosive means prior to concrete placement. Grouting and internal drainage were not shown on the record drawings. Data on structural stability were not available.

6.1.3 Operating Records: Records show that the maximum water level at the weir was 303.9 ft.

6.1.4 Post Construction Changes: There is no indication of significant post-construction changes in this dam other than the addition of the permanent stoplogs which raised the pool by two feet.

6.1.5 Seismic Stability: The dam is located within Zone 1 on the Algermissens Seismic Risk Map (1969) of the United States. Calculations indicate it does not meet the screening criteria for a conventional margin of safety against overturning (see paragraph 6.2 below) and therefore in accordance with paragraph 3.6.4 of Reference 1, seismic stability studies should be made.

6.2 Calculation Results: Calculations based on the PMF, 0.5 PMF, and normal operating level with ice and 100 percent uplift at the upstream toe, indicate that the structure has an adequate factor of safety against sliding and overstressing, and is stable in that the righting moments are at least 13 percent greater than the overturning moments under the most severe (PMF) design conditions, however because the force resultant does not lie in the middle third of the dam base, it cannot be considered to have a conventional margin of safety with respect to overturning according to the screening criteria established by the Corps of Engineers (Paragraph 4.4.4.4 of Reference 1). For other details see Appendix H.

6.3 Concrete: According to the 1925 Commissioner's report (Reference 3), the concrete for contract 7 which included the Overflow Weir was mixed at a new plant built several hundred feet north of the Weir. On page 154 of Reference 3 it states: "The concrete for the masonry work on this contract is proportioned in accordance with A. Abrams' method. Laboratory tests are made of the gradation and other qualities of the aggregates and a mix is designed on the basis of these tests which will give a predetermined strength. The mix used in the core-walls is one part of cement to seven parts of aggregate. The percentages of sand and gravel are varied in accordance with the variations in gradations of these ingredients as developed by control analyses of the bank-run material made from time to time. Compressive test specimens of the mixes designed are made in the laboratory and, in addition, test specimens are made at frequent intervals from concrete taken from the forms. This aids materially in obtaining a uniform product."

Precautions taken during winter reportedly included heating the aggregate and water, plus steam curing the concrete when fresh.

7.0 ASSESSMENT RECOMMENDATIONS/REMEDIAL MEASURES

The assessment, recommendations and remedial measures contained herein are based on the provisions of Appendix J, Conditions.

7.1 DAM ASSESSMENT: On the basis of the visual field inspection and available engineering, operational, and performance data, the dam does not now exhibit critical signs of distress such as severe structural cracking, abnormal movement, severe leakage or seepage, or unstable abutment conditions. Nevertheless, the evidence listed below cause a necessary concern regarding the long term serviceability of the Overflow Weir.

a. The time required to draw the reservoir down to the base of the Overflow Weir using the existing facilities for Wanaque Reservoir is considered excessive. Adequate drawdown capacity may be necessary to reduce damage to a failing structure or to enable repairs on the reservoir side.

b. There is some minor undermining under the downstream left abutment wall.

c. There is seepage (2 to 5 gpm) at the downstream left retaining wall.

d. The dam does not have a conventional margin of safety against overturning.

e. The earth fills adjacent to the concrete weir structure will be overtopped under PMF conditions, possibly creating an additional flood hazard.

f. There is surface cracking of the gunite and an area of severe scaling on the left downstream retaining wall and a similar condition on several piers.

7.2 RECOMMENDATIONS/REMEDIAL MEASURES: The following measures are recommended because of the previously stated concerns:

a. A means of reducing the time required to lower the water level in the reservoir below the stoplogs should be studied soon and provided in the near future.

b. The scour area should be protected from additional scour and the undermined area repaired soon.

c. The wall seepage should be closely and regularly monitored starting very soon and the source of the seepage should be determined soon.

d. The dam should have a seismic stability analysis performed in accordance with paragraph 4.4.2.1 of Reference 1 in the near future.

e. Future studies should be performed to determine the feasibility of providing additional flood discharge facilities elsewhere on the reservoir, or revising the overflow elevation of the Weir to provide additional flood storage. This should be done in conjunction with a thorough review of what the design flood must be for this reservoir.

f. Areas of deteriorated concrete should be rehabilitated in the future.

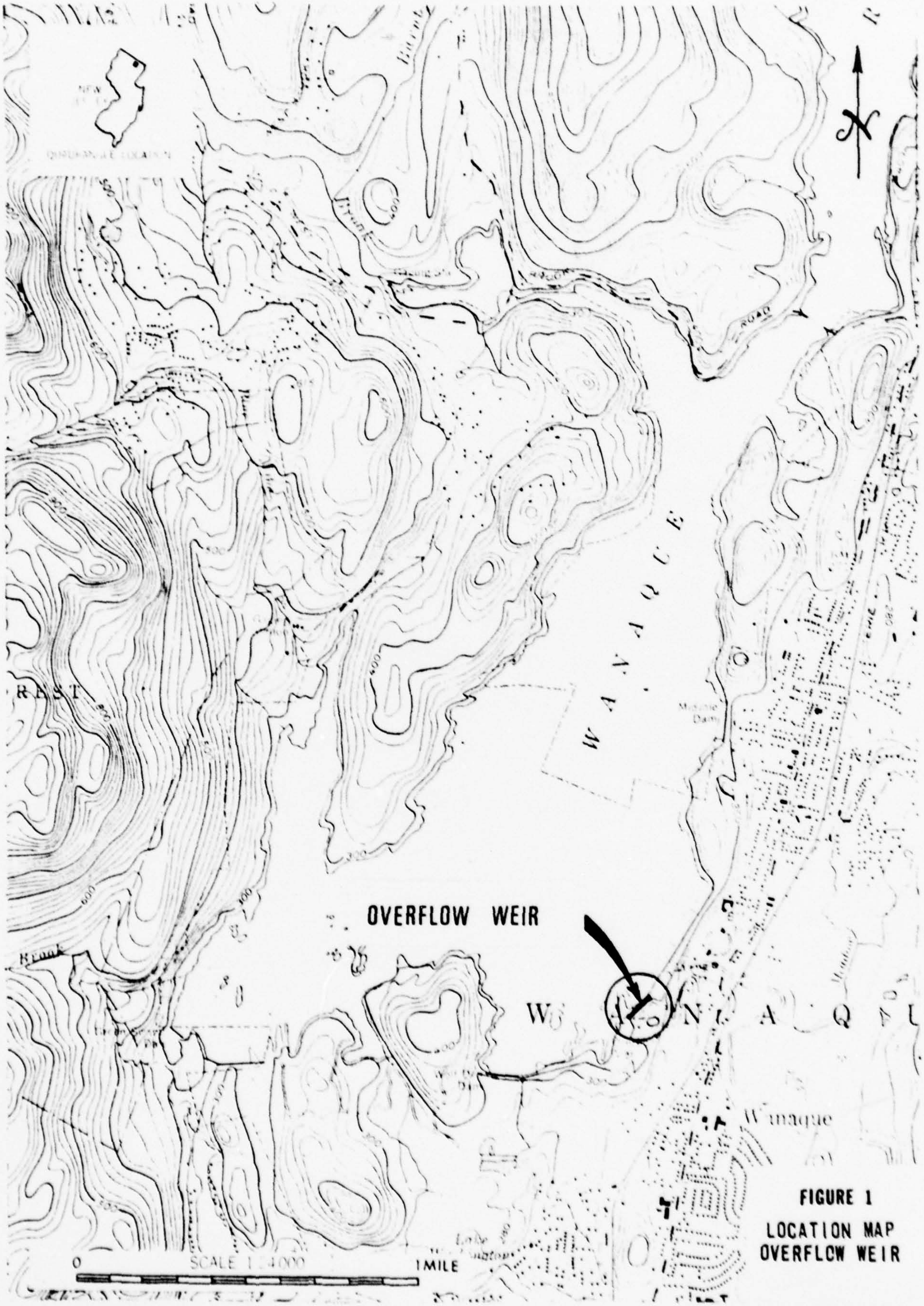
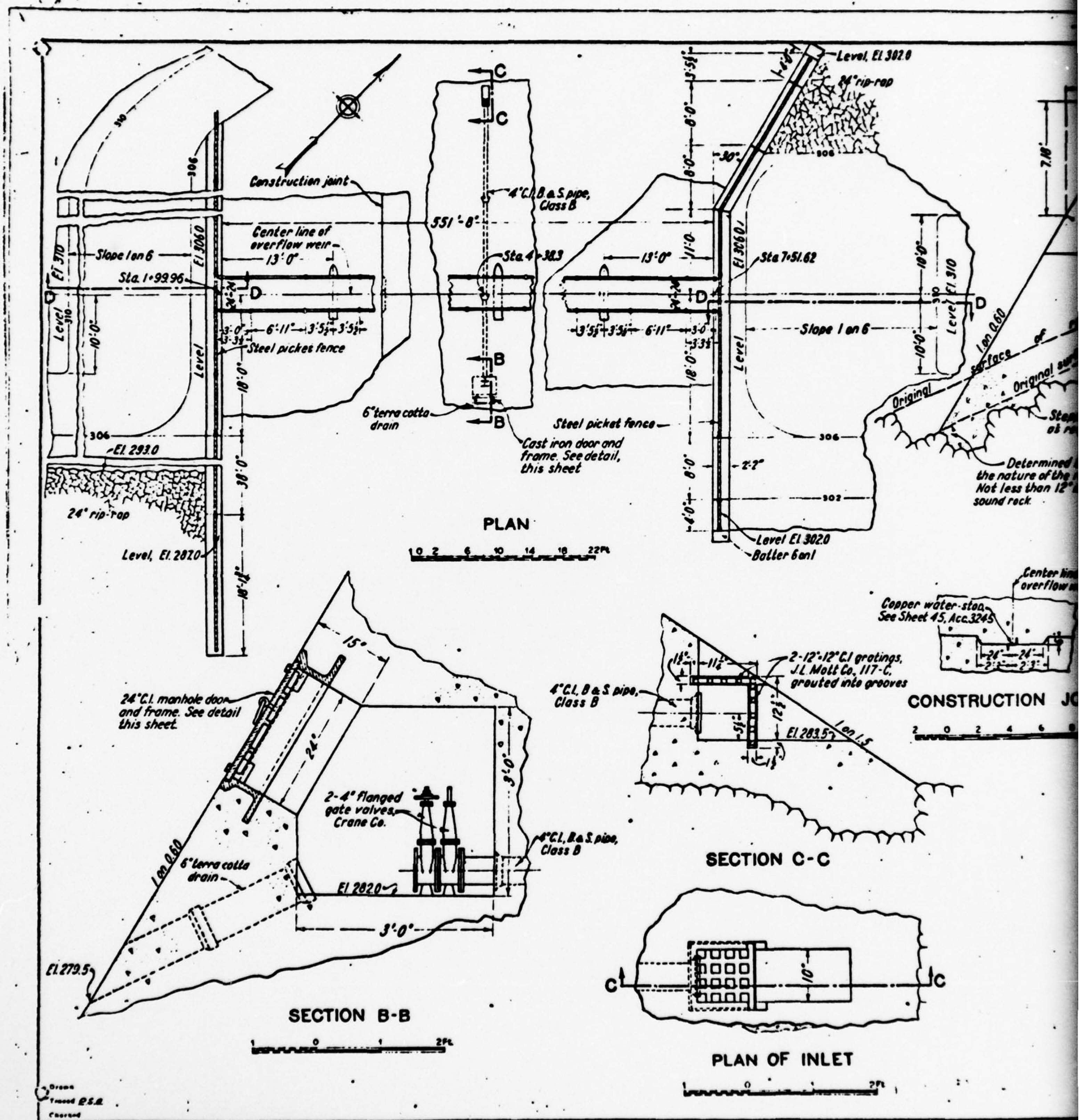
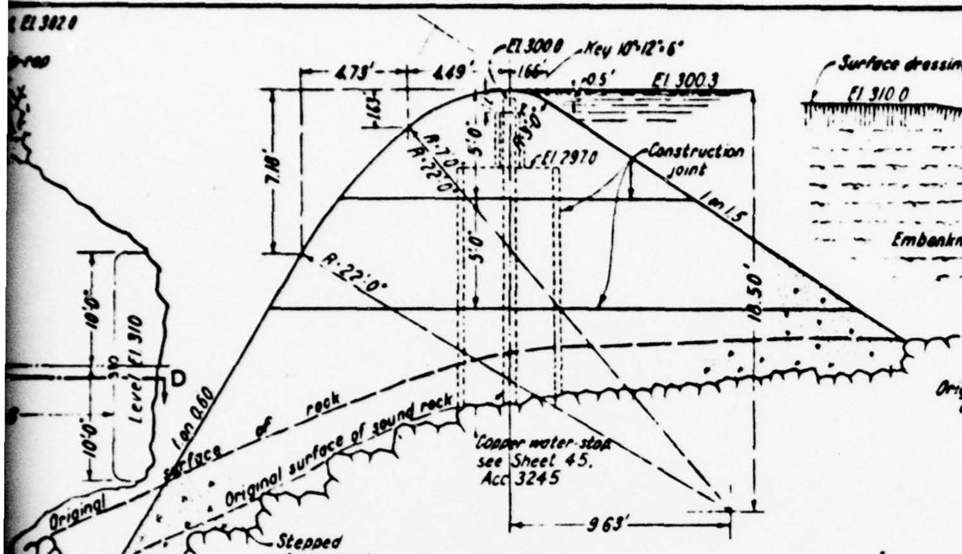


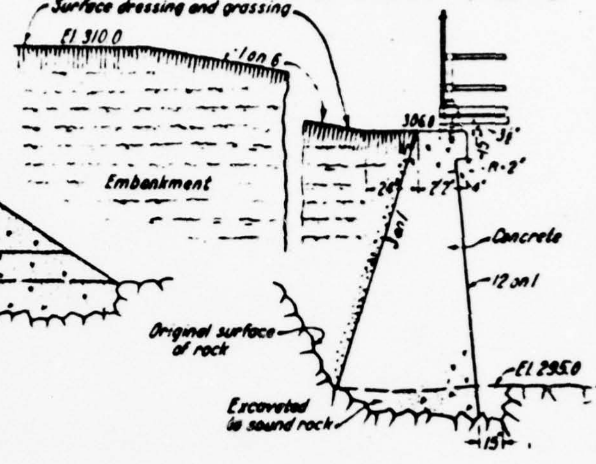
FIGURE 1
LOCATION MAP
OVERFLOW WEIR


PLAN
Construction joint
Center line of overflow weir
Slope 1 on 6
Sta. 1+99.96
El. 306.0
El. 310
Level 310
Level
Steel picket fence
El. 293.0
24" rip-rap
Level, El. 287.0
4" C.I. B. & S. pipe, Class B
551'-8"
Sta. 4+38.3
6" terra cotta drain
Cast iron door and frame. See detail, this sheet
Steel picket fence
Level, El. 302.0
24" rip-rap
Sta. 7+51.62
Level El. 302.0
Batter 6 on 1
Original surface of rock
Determined by the nature of the rock. Not less than 12" in sound rock.
SECTION B-B
24" C.I. manhole door and frame. See detail this sheet.
2-4" flanged gate valves, Crane Co.
4" C.I., B & S. pipe, Class B
6" terra cotta drain
1 on 0.60
El. 282.0
El. 279.5
SECTION C-C
4" C.I. B & S. pipe, Class B
2-12"×12" C.I. gratings, J.L. Mott Co. 117-C, grouted into grooves
El. 283.5
1 on 1.5
PLAN OF INLET
Copper water-stop. See Sheet 45, Acc. 3245
CONSTRUCTION JO
Drawn
Traced R.S.B.
Checked

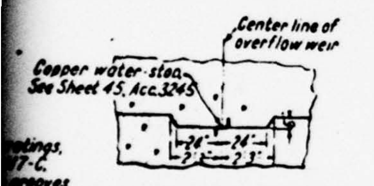


SECTION A-A

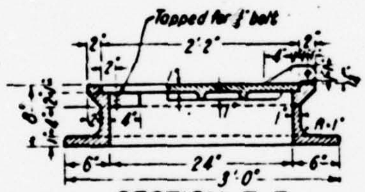
RECORD DRAWING
SHEET 37 SHEETS IN SET 61



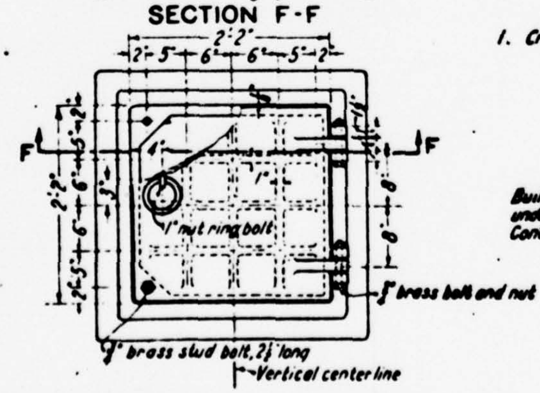
SECTION D-D



CONSTRUCTION JOINT



SECTION F-F



CAST IRON DOOR AND FRAME

REFERENCES

1. For detail of steel picket fence see Sheet 38, Acc 3238.
2. For elevations of retaining walls at ends of weir see Sections E-E and F-F, Sheet 38, Acc 3238.

NOTE

1. Crest of weir does not vary more than ½" from El 300.3

CONSTRUCTION RECORD

Built September 10 1926 to April 10 1928 under Contract 7, Clifford F. MacEvoy Co., Newark, N.J. Contractor, as shown on this record drawing.

Engineer in charge.

NORTH JERSEY DISTRICT
WATER SUPPLY COMMISSION

WANAQUE RESERVOIR

OVERFLOW WEIR

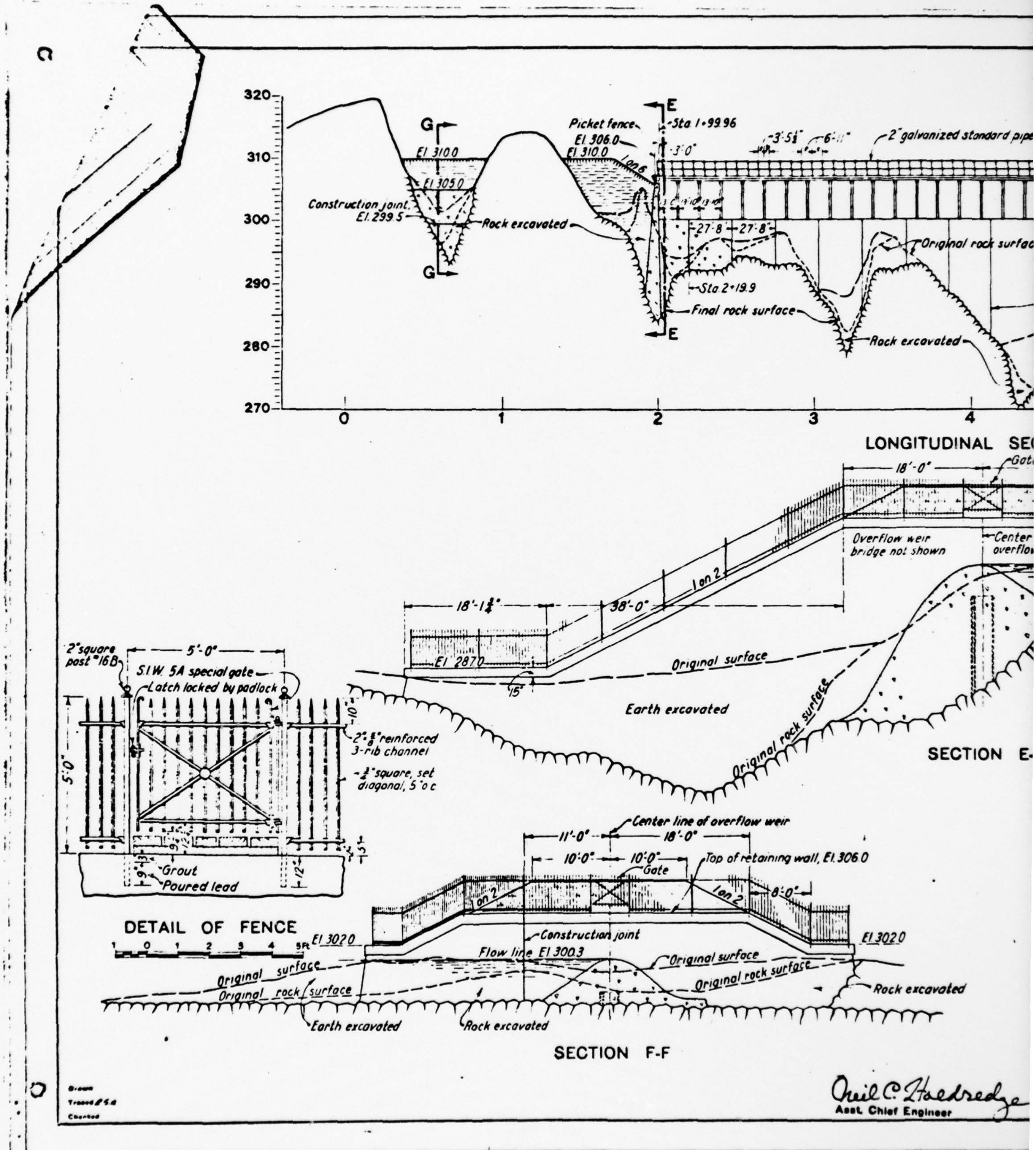
PLAN AND SECTIONS

Neil C. Hredredge
Asst. Chief Engineer

APRIL 30, 1931

FIGURE 2

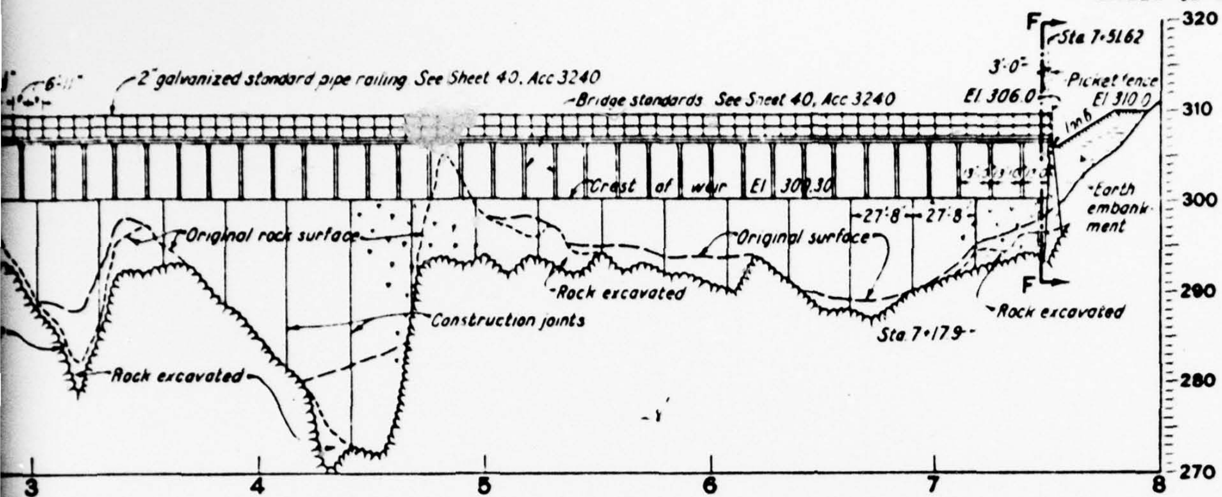
Case C Dr. 12   File 3.44 W   Acc. 3237



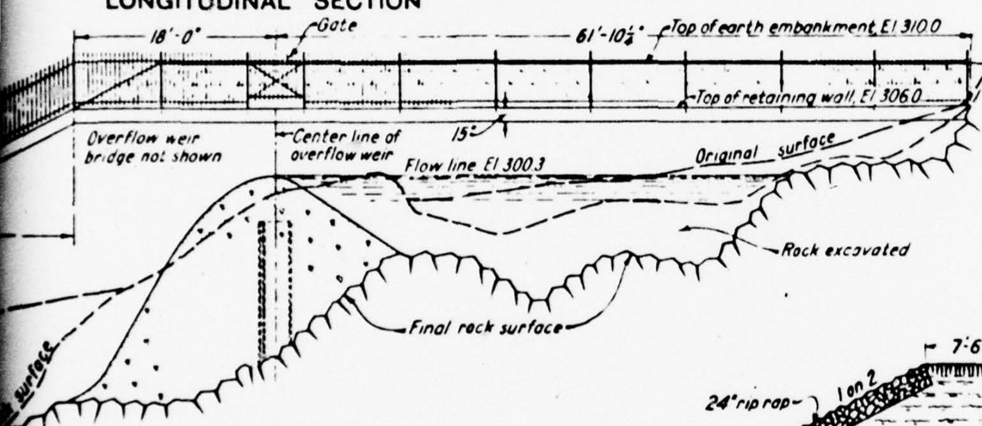
320
310
300
290
280
270
0
1
2
3
4
G
E
Picket fence
Sta 1+99.96
El 306.0
El 310.0
El 310.0
3'-0"
3'-5½"
6'-11"
2" galvanized standard pipe
El 305.0
Construction joint.
El 299.5
Rock excavated
27'-8"
27'-8"
Original rock surface
Sta 2+19.9
Final rock surface
Rock excavated
LONGITUDINAL SEC
18'-0"
Gate
Overflow weir bridge not shown
Center overflow
18'-1½"
38'-0"
1 on 2
El 287.0
15"
Original surface
Earth excavated
Original rock surface
SECTION E-
2" square post #16B
5'-0"
S.I.W. 5A special gate
Latch locked by padlock
2" x ⅝" reinforced 3-rib channel
½" square, set diagonal, 5" o c
Grout
Poured lead
DETAIL OF FENCE
Center line of overflow weir
11'-0"
18'-0"
10'-0"
10'-0"
Gate
Top of retaining wall, El. 306.0
1 on 2
8'-0"
El 302.0
El 302.0
Construction joint
Flow line El 300.3
Original surface
Original surface
Original rock surface
Original rock surface
Rock excavated
Earth excavated
Rock excavated
SECTION F-F
Asst. Chief Engineer

Figure 2

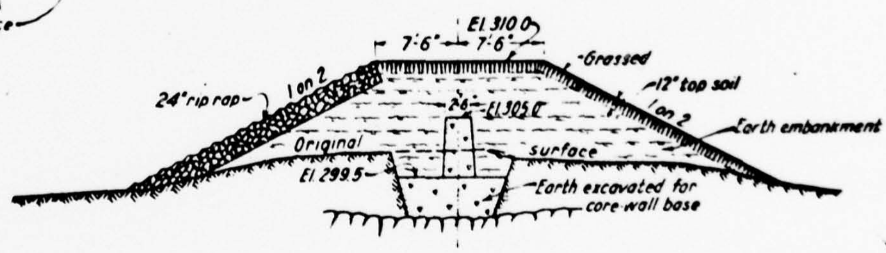
RECORD DRAWING
SHEET 38 SHEETS IN SET 61



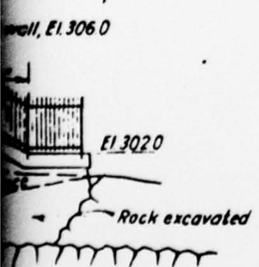
LONGITUDINAL SECTION



SECTION E-E



SECTION G-G



REFERENCE

For plan and additional sections see Sheet 37, Acc 3237

NOTES

1. *Picket fence furnished by Stewart Iron Works Co (American Fence Construction Co, 221 W 57 St., New York City), from whose catalogue No. 60A, the designations shown on "Detail of Fence", are taken.*
2. *All material heavily galvanized after fabrication.*

CONSTRUCTION RECORD

Built September 10, 1926 to June 25, 1928, under Contract 7, Clifford F. MacEvoy Co., Newark, N.J., Contractor, as shown on this record drawing.

Neil C. Holdredge
Asst. Chief Engineer

. *Engineer in charge.*

NORTH JERSEY DISTRICT
WATER SUPPLY COMMISSION

WANAQUE RESERVOIR
OVERFLOW WEIR
SECTIONS

0 2 6 ... 30

APRIL 30, 1931

FIGURE 3

CASE C DR. 12 File-3.44 W Acc. 3238

2

APPENDIX A

VISUAL CHECKLIST

APPENDIX A - VISUAL INSPECTION CHECK LIST
PHASE 1

Name Dam: Overflow Weir   County: Passaic   State: New Jersey   Coordinators: Philadelphia District-Corps of Engineers

Date(s) Inspection: 9-10, and 23 May, 1978   Weather: Clear   Temperature: 70°

Pool Elevation at Time of Inspection: 301.5 & 302.4 M.S.L.   Tailwater at Time of Inspection: 280.0 and 283.0 MSL

Inspection Personnel*	Also Present:
Fine T. Hsu	Mario DiLaura (NJDWSC)
James A. Hagen (Not 23 May)	Larry Woscyna (NJDEP)
Rudolph J. Wahanik	Robert Wieland (NJDWSC) - Part Time

*Gilbert Associates, Inc.

James A. Hagen - Recorder

CONCRETE/MASONRY DAMS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATION
SEEPAGE OR LEAKAGE	Approximately 2 gpm of seepage occurred at the downstream spalling and cracked face of the left concrete retaining wall, about 7 feet-8 feet beyond the toe (inspection date: 5/23/78, pool level @ el. 302.4).	The seepage condition should be closely and regularly monitored. The source of seepage should be further determined and corrected accordingly.
STRUCTURE TO ABUTMENT/EMBANKMENT FUNCTIONS	The left retaining wall foundation was partially undermined by scouring.	The retaining wall foundation should be protected from damage by scouring. The undermined portion should be repaired.
DRAINS	The manhole door covering the four inch drain valves (for draining an upstream low area) was bolted shut. No leakage was evident from the six inch drain leaving the area.	This drain would be used only to completely drain the low area against the dam, in the event of total drawdown of the reservoir.
WATER PASSAGES	None.	
FOUNDATION	The overflow concrete weir was built across the irregular rock ledge area. The foundation rocks are primarily gneisses, hard and competent when unweathered. The rock foliation planes strike N 23-30° E 50-60° SE. No fault or shear planes were observed in the rocks.	A continuous exposure of irregular rock masses at and beyond the toe should provide a large passive resistance against foundation sliding forces.
SURFACE CRACKS CONCRETE SURFACES	A portion of the concrete surface of the left retaining wall showed severe scaling and surface cracking.	The deteriorated concrete needs to be rehabilitated.
STRUCTURAL CRACKING	None evident.	The gunite resurfacing makes identification of older, dormant cracks nearly impossible.

CONCRETE/MASONRY DAMS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATION
VERTICAL AND HORIZONTAL ALIGNMENT	The vertical and horizontal alignment of the weir appears to be very good.	
MONOLITH JOINTS	The visible vertical joints appeared to be in satisfactory condition with no excessive spalling.	
CONSTRUCTION JOINTS	Not evident due to gunite resurfacing.	

EMBANKMENT

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATION
SURFACE CRACKS	Not applicable.	
UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE	Not applicable.	
SLOUGHING OR EROSION OF EMBANKMENT AND ABUTMENT SLOPES	Not applicable.	
VERTICAL AND HORIZONTAL ALINEMENT OF THE CREST	Not applicable.	
RIPRAP FAILURES	Not applicable.	
JUNCTION OF EMBANKMENT AND ABUTMENT, SPILLWAY AND DAM	Not applicable.	
ANY NOTICEABLE SEEPAGE	Not applicable.	
STAFF GAGE AND RECORDER	Not applicable.	
DRAINS	Not applicable.	

OUTLET WORKS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATION
CRACKING AND SPALLING OF CONCRETE SURFACES IN OUTLET CONDUIT	Not applicable.	
INTAKE STRUCTURE	Not applicable.	
OUTLET STRUCTURE	Not applicable.	
OUTLET CHANNEL	Not applicable.	
EMERGENCY GATE	Not applicable.	

UNGATED SPILLWAY

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATION
CONCRETE WEIR	The gunite surface has raised the concrete crest elevation about 0.2 feet. The surface is generally smooth with no significant signs of deterioration. Three stoplogs are "permanently" installed.	
APPROACH CHANNEL	None	
STILLING BASIN AND DISCHARGE CHANNEL	An excellent natural rock basin was formed (after cleaning out pocket sediments) and was used as a stilling basin below the spillway. The discharge channel is deeply cut through the hard rock masses with steep but stable side slopes.	
BRIDGE AND PIERS	A foot bridge, with piers set in the concrete mass of the weir, appears to be in good condition. A few pieces of wooden deck board were missing at the time of the inspection. Several piers have small areas of very severe scaling of the concrete to a maximum depth of 0.2 feet.	The scaling of the piers is not located in such a position that it would affect the functioning of the main structure; however, the areas of scaling should be repaired in the future to prevent further deterioration.

GATED SPILLWAY - (NONE WAS OBSERVED)

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATION
CONCRETE SILL	Not applicable.	
APPROACH CHANNEL	Not applicable.	
DISCHARGE CHANNEL	Not applicable.	
BRIDGE AND PIERS	Not applicable.	
GATES AND OPERATION EQUIPMENT	Not applicable.	

INSTRUMENTATION

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATION
MONUMENTATION/SURVEYS	None were observed.	
OBSERVATION WELLS	None.	
WEIRS	None.	
PIEZOMETERS	None.	
OTHER		

RESERVOIR

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATION
SLOPES	The slopes of the reservoir rim near the Weir vary from gentle along the left bank to steep on the right bank. The Precambrian gneisses are continuously exposed along the slopes and maintain a stable slope condition.	None
SEDIMENTATION	Because of this the geologic environment of the Precambrian Highlands, a thin to non-existent soil cover, and a good vegetation cover around this part of the reservoir, the sedimentation process is being carried on at an extremely slow rate. No sediments were visible around the Weir.	None

APPENDIX B

ENGINEERING DATA CHECKLIST

APPENDIX B
CHECK LIST
ENGINEERING DATA
DESIGN, CONSTRUCTION, OPERATION

ITEM	REMARKS
PLAN OF DAM	A tracing of the record drawing is available at the North Jersey District Water Supply Commission (NJDWSC) office in Wanauqe, N.J. (hereafter referred to as NJDWSC-W)
REGIONAL VICINITY MAP	The USGS Wanaque, N.J. 7-1/2 min. quadrangle map is available.
CONSTRUCTION HISTORY	The 1925 Commissioner's Report (Reference 3) is available at NJDWSC-W. There is also a 1931 Commissioner's report at NJDWSC-W, an article on the construction was printed in the N.E.W.W.A. Journal (Reference 2) during construction. Some photos are available in the NJDWSC-W and the N.J. Dept. of Environmental Protection offices in Trenton, N.J. (DEP).
TYPICAL SECTIONS OF DAM	A section through the dam is shown on record drawings No. 37 and 38 of 61 which are available at NJDWSC-W. (See Figures 2 and 3)
HYDROLOGIC/HYDRAULIC DATA	Records are available at NJDWSC-W and some are printed in USGS reports.
OUTLETS - PLAN	There are no other significant outlets at this dam.
- DETAILS	
- CONSTRAINTS	
- DISCHARGE RATINGS	
RAINFALL/RESERVOIR RECORDS	Excellent records are available from the USGS and NJDWSC beginning at the time of construction of this dam.

ITEM	REMARKS
DESIGN REPORTS	A single design report was not made; however, a brief description of the dam design is shown in the NJDWSC's 1925 report, Chapter 11, pages 44-48.
GEOLOGY REPORTS	Geological reports of this dam are not available. A geological map covering this site on a scale of 1 inch to one mile published by the New Jersey Geological Survey is attached as Appendix F.
DESIGN COMPUTATIONS	None available
HYDROLOGY & HYDRAULICS	Incomplete
DAM STABILITY	None available
SEEPAGE STUDIES	None available.
MATERIALS INVESTIGATIONS	
BORING RECORDS	
LABORATORY	
FIELD	Foundation test borings and grouting records were not available.
POST-CONSTRUCTION SURVEYS OF DAM	Available record drawings may be the results of post-construction survey.
BORROW SOURCES	Locations and logs of borrow areas are shown on contract drawings available at NJDWSC-W.
SPILLWAY PLAN	
SECTIONS DETAILS	Overflow sections and details are shown on Record Drawings Sheet Nos. 37 and 38 in set 61 at the NJDWSC-W office. (See Figures 2 and 3 of text)
OPERATING EQUIPMENT PLANS & DETAILS	None

APPENDIX B - CONT'D

ITEM	REMARKS
MONITORING SYSTEMS	None observed.
MODIFICATIONS	Provision was made in the original design for the later addition of stoplogs. Other than the addition of three permanent stoplogs and surface guniting, there were no modifications observed.
HIGH POOL RECORDS	Records exist at the NJDWSC-W and USGS publications.
POST CONSTRUCTION ENGINEERING STUDIES AND REPORTS	Annual reports for certain years are in dam file No. 32 of DEP.
PRIOR ACCIDENTS OR FAILURE OF DAM DESCRIPTION REPORTS	None reported nor suspected.
MAINTENANCE OPERATION RECORDS	Operational levels of the reservoir are available from NJDWSC-W.

APPENDIX B - CONT'D

CHECK LIST
ENGINEERING DATA
HYDROLOGIC AND HYDRAULIC DATA

DRAINAGE AREA CHARACTERISTICS: Densely forrested, very hilly with minimal cover on bedrock.

ELEVATION TOP NORMAL POOL (STORAGE CAPACITY): 302.4

ELEVATION TOP FLOOD CONTROL POOL (STORAGE CAPACITY): Not Available

ELEVATION MAXIMUM DESIGN POOL: 308.8

ELEVATION TOP DAM: 302.4

CREST:

a. Elevation: 302.4
b. Type: Ogee Spillway
c. Width: The crest of the spillway is curvilinear.
d. Length: 552 (total distance from abutment to abutment)
e. Location Spillover: The entire structure serves as a spillway.
f. Number and Type of Gates: None

OUTLET WORKS: None

a. Type: Not Applicable
b. Location: Not Applicable
c. Entrance inverts: Not Applicable
d. Exit inverts: Not Applicable
e. Emergency draindown facilities: Not Applicable

HYDROMETEOROLOGICAL GAGES:

a. Type: Precipitation cans, water level recording chart, streamflow recording chart and high and low daily temperature recorder.
b. Location: At Raymond Dam, about 0.3 miles northeast.
c. Records: Streamflow published by USGS. Precipitation and temperature published by the U.S. Weather Service.

MAXIMUM NON-DAMAGING DISCHARGE: For the existing spillway, the maximum non-damaging water elevation is approximately 306.00 ft. That corresponds to a discharge of a) 14,200 cfs with flashboards. b) 27,300 cfs without flashboards.

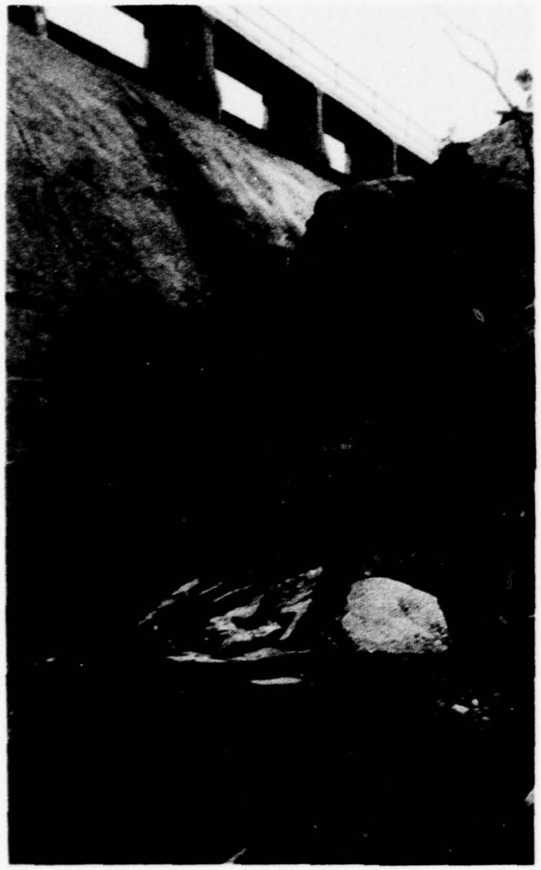
APPENDIX C

PHOTOGRAPHS

DISCHARGE POINT - STILLING
BASIN TO DISCHARGE CANAL
MAY 1978

May 1978

LOWER LEFT ABUTMENT RETAINING WALL SHOWING SURFACE
CRACKING OF GUNNITE, AREAS OF SEVERE SCALING OF CONCRETE,
AND VOIDS UNDER WALL



May 1978

MAXIMUM SECTION - NOTE BLOWOFF

May 1978

DETAIL AT STOPLOGS AND SURFACE
DETERIORATION OF CONCRETE

APPENDIX D

RESERVOIR HYDROLOGY AND DRAWDOWN

APPENDIX D

RESERVOIR HYDROLOGY AND DRAWDOWN

Reservoir Hydrology

The hydrologic analysis presented in this Report and in the Appendix pertains to present hydrologic conditions and does not consider future changes produced by uncertain conditions such as urbanization, forest fires, or other modifications within the watershed.

The inflow probable maximum flood (PMF) hydrograph for Wanaque Reservoir was supplied by the Philadelphia Office of the Corps of Engineers (Reference 8) and is shown in Figure D-1. This hydrograph has a peak flow rate of 33,500 cfs occurring 50 hours after its start. The total runoff volume is 94,500 acre-feet, over a time span of 140 hours. The HEC-1 computer program (Reference 9) was used to route this hydrograph through the reservoir. The main discharge structure for Wanaque Reservoir is a 520-foot long overflow weir which has had permanent flashboards in place since 1934. The storage volume-spillway outflow relation was determined assuming that the initial water surface elevation was at the top of the flashboards (302.4) and the structure functions as a sharp-crested weir.

Because the flashboards are not designed to break away, the spillway discharge and the reservoir storage/spillway outflow relationship used in HEC-1 for routing the PMF and one-half the PMF through the reservoir assume the flashboards are in place. These relationships are in Figure D-2.

Water Elevation ft.	Spillway Discharge cfs	Reservoir Storage Acre-ft.
302.4	0	0
303	820	1381
304	3760	3530
305	8410	5678
306	14210	7765
307	18640	9822
308	23700	12431
309	28900	14270
310	35300	16418

The surface area and storage of the Wanaque Reservoir at different water levels (Reference 2) are shown in Figure D-3. Their values are:

Water Elevation ft.	Surface Area Acre	Storage Acre-ft.
215	0	0
220	40	153
230	190	1228
240	370	4910
250	790	9820
260	1070	19027
270	1300	31303
280	1630	45420
290	1960	63326
300	2310	84701
310*	2620	106183
312*	2680	110480

*Values extrapolated from elevation 305.00 ft. (Reference 2).

Results of this routing procedure indicate that the PMF would raise the pool elevation to about 308.8 feet. Routing one-half the PMF (16,750 cfs) through Wanaque Reservoir raises the pool elevation to about 306.0 ft.

Flood routing was also performed assuming that the flashboards were removed. In this case, the storage volume-outflow relation was determined with the starting water surface elevation at the top of the spillway crest (300.3 feet.) and the Overflow Weir discharging as an uncontrolled ogee crest spillway. HEC-1 results indicate that the PMF would raise the pool elevation to 306.9 feet. The reservoir was designed to safely discharge 18,000 cfs (slightly larger than one-half the PMF) without the flashboards in place. Graphs of pool elevation versus time for the PMF and one-half the PMF routing, with and without flashboards, are found in Figures D-4 and D-5.

A summary of the flood routing flows through the Wanaque Reservoir and the corresponding water levels is given below:

a. With Flashboards

Flood Description	Inflow Peak cfs	Outflow Peak cfs	Pool Elevation ft	Head Above Weir Crest ft
PMF	33500	27900	308.8	6.40
One-half PMF	16750	14000	306.0	3.60

b. Without Flashboards

Flood Description	Inflow Peak cfs	Outflow Peak cfs	Pool Elevation ft	Head Above Weir Crest ft
PMF	33500	29100	306.3	6.00
One-half PMF	16750	13800	303.9	3.60

Reservoir Drawdown

If an emergency condition develops that affects the stability of one of several dams that form the Wanaque Reservoir or of the outlet and control works of the Raymond Dam, then a fast drawdown of the reservoir to a lower water level will be required. The lower water level depends on the location and nature of the hazardous condition. Figure D-6 shows graphically the times required to lower the reservoir level with the existing facilities.

The water level in the Wanaque Reservoir can be lowered by means of:

a. The Wanaque Aqueduct System.

b. The existing aerator system.

c. A 36 inch diameter blowoff.

d. The blowoff and the aerator together.

e. Other blowoff lines.

All drawdown times were computed considering that the minimum inflow of 2 cfs/square mile into the reservoir was equalized by the system demand and other water loses.

A. The Wanaque Aqueduct

The potential of the Wanaque Aqueduct to lower the water level in the reservoir during an emergency condition is non-existent because a minimum inflow of 2 cfs/square mile, which is equivalent to 117 MGD, will supply the average daily demand of the distribution system. Table 1 gives the average water consumption during the last 10 years.

Table 1

Year	Demand (MGD)
1967	95.37
1968	106.92
1969	111.17
1970	113.45
1971	112.88
1972	112.17
1973	103.09
1974	98.90
1975	92.07
1976	90.58
1977	107.90

B. Aerator System

Operation of the existing aerator system will drawdown the reservoir water level between the crest of the Overflow Weir at elevation 302.4 feet and the top of the aeration nozzles at elevation 240.5 feet in the following times:

Water Level (Feet)	Total Time (Days)
302.4	0
300	11
290	56
280	98
270	136
260	175
250	213
240.5	254

C. 36-Inch Diameter Blowoff

The 36-inch diameter blowoff installed at the bottom of the Raymond Dam in the stream control conduits can be used to lower the reservoir level to an elevation of 222.00 which corresponds to the entrance intake fill to the lower conduit. The blowoff discharge is through an 18-inch diameter pipe that has two valves, one 18-inch valve and one 8-inch valve, both located at centerline elevation 213.38 feet. The times in days required by the blowoff line operating alone to lower the reservoir water level through 18-inch diameter valve is:

Water Level (Feet)	Total Time (Days)
302.4	0
300	34
290	178
280	310
270	421
260	526
250	614
240	678
230	714
222	729

D. Blowoff and Aerator

Simultaneous operation of the 36-inch diameter blowoff pipe in conjunction with the aerator system will lower the reservoir water level in the following times:

Water Level (Feet)	Total Time (Days)
302.4	0
300	8
295	43
280	75
270	103
260	131
250	158
240	182
230	219
222	234

E. Other Blowoff Lines

Smaller diameter blowoff lines installed in several of the dams around the Wanaque Reservoir are not known to be in operable condition because, since its installation in 1925, the lines have not been inspected, operated, or maintained.

DEPARTMENT OF THE ARMY
PHILADELPHIA DISTRICT, CORPS OF ENGINEERS
CUSTOM HOUSE – 2 D & CHESTNUT STREETS
PHILADELPHIA, PENNSYLVANIA 19106

IN REPLY REFER TO
NAPEN-H

12 MAY 1978

Mr. Robert A. Putt
Hydraulic Engineer
Gilbert Associates, Inc.
P. O. Box 1498
Reading, PA 19603

Dear Mr. Putt:

The following information is to be applied when determining the Spillway Design Flood for the first nine dams Gilbert/Commonwealth is inspecting in connection with the Dam Safety Program.

For the five dams around the Wanaque Reservoir (Raymond, Wolf Den, Furnace Road, Midvale and the Overflow Weir), the hydrograph used should conform to the data specified below:

Peak Q= 33,500 cfs

Shape should be similar to the PMF hydrograph labeled 'Wanaque River at Mouth MPF=29,300 cfs SPF = 14,000 cfs shown on Figure A63 of the Passaic River Basin Report. (A copy of this figure is inclosed)

The above information is from the Passaic River Basin-New Jersey and New York Survey Report for Water Resources dated June 1972 by the New York District Corps of Engineers. The drainage area above the Wanaque Dam stated in this report is slightly different than the drainage area you have supplied to us in your letter dated 9 May 1978. To maintain consistency between reports we would suggest using 90.4 sq. mi. for the drainage area above the Wanaque Reservoir as published in Table A1 of the Passaic River Basin Report.

Due to the small drainage area of the remaining four dams (Glen Wild Lake -1.04 sq. mi., Lake Vreeland-0.83 square mi., Crystal Lake-4.34 sq. mi., and Cedar Grove Reservoir-0.45 Sq. mi.), the hydrographs for these dams should be developed using the SCS triangular method. For an example of this method see Pg. 74 of Design of Small Dams (second edition) by the U. S. Department of Interior-Bureau of Reclamation.

NAPEN-H
Mr. Robert A. Putt

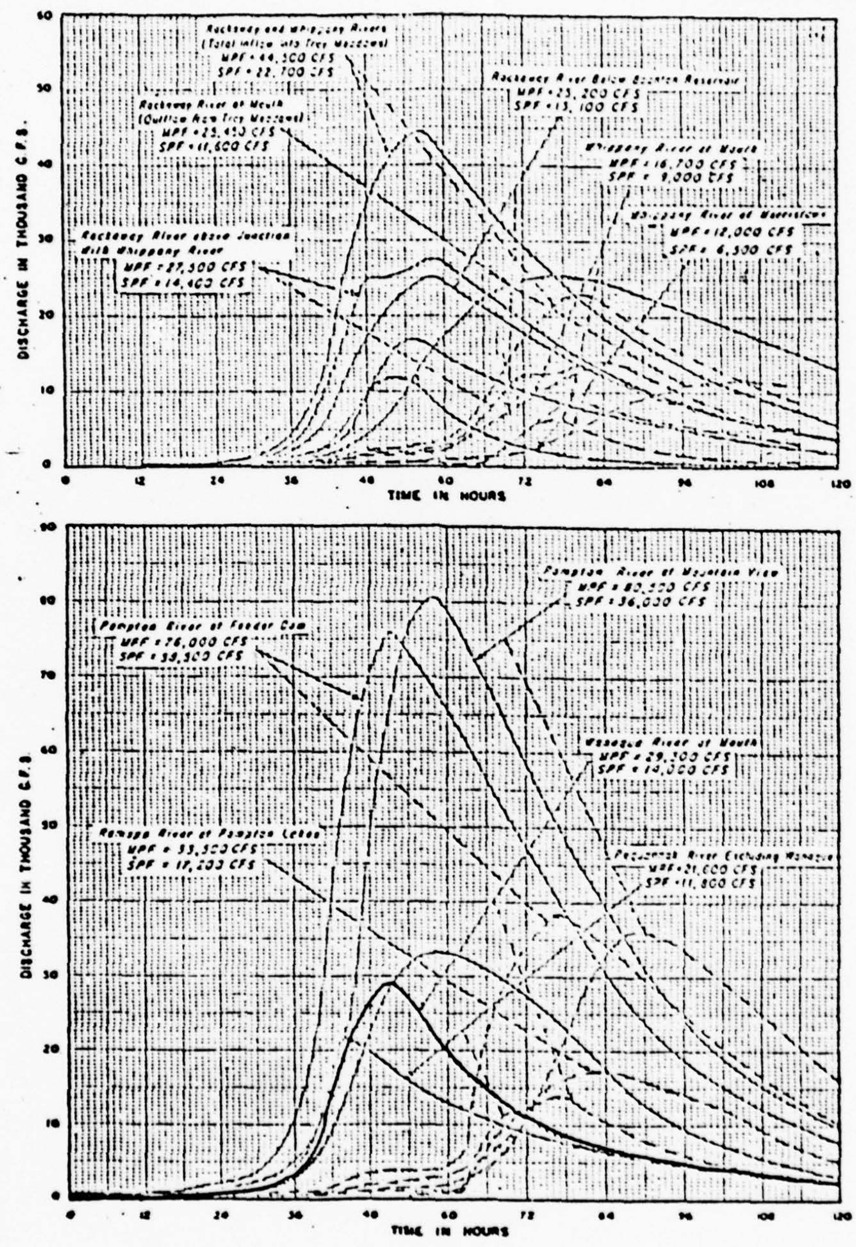
If there are any other questions, please do not hesitate to contact us.

Sincerely yours,

Leonard J. Lipski

1 Incl
Fig. A63 (Passaic River Basin Report)

LEONARD J. LIPSKI
Chief, Hydrology-Hydraulics Branch



PASSAIC RIVER BASIN, N. J. AND N. Y.

STANDARD PROJECT AND MAXIMUM PROBABLE FLOODS (EXISTING CONDITIONS WITH RAINFALL OVER ENTIRE BASIN ABOVE PATERSON, N.J.)

FIGURE A63

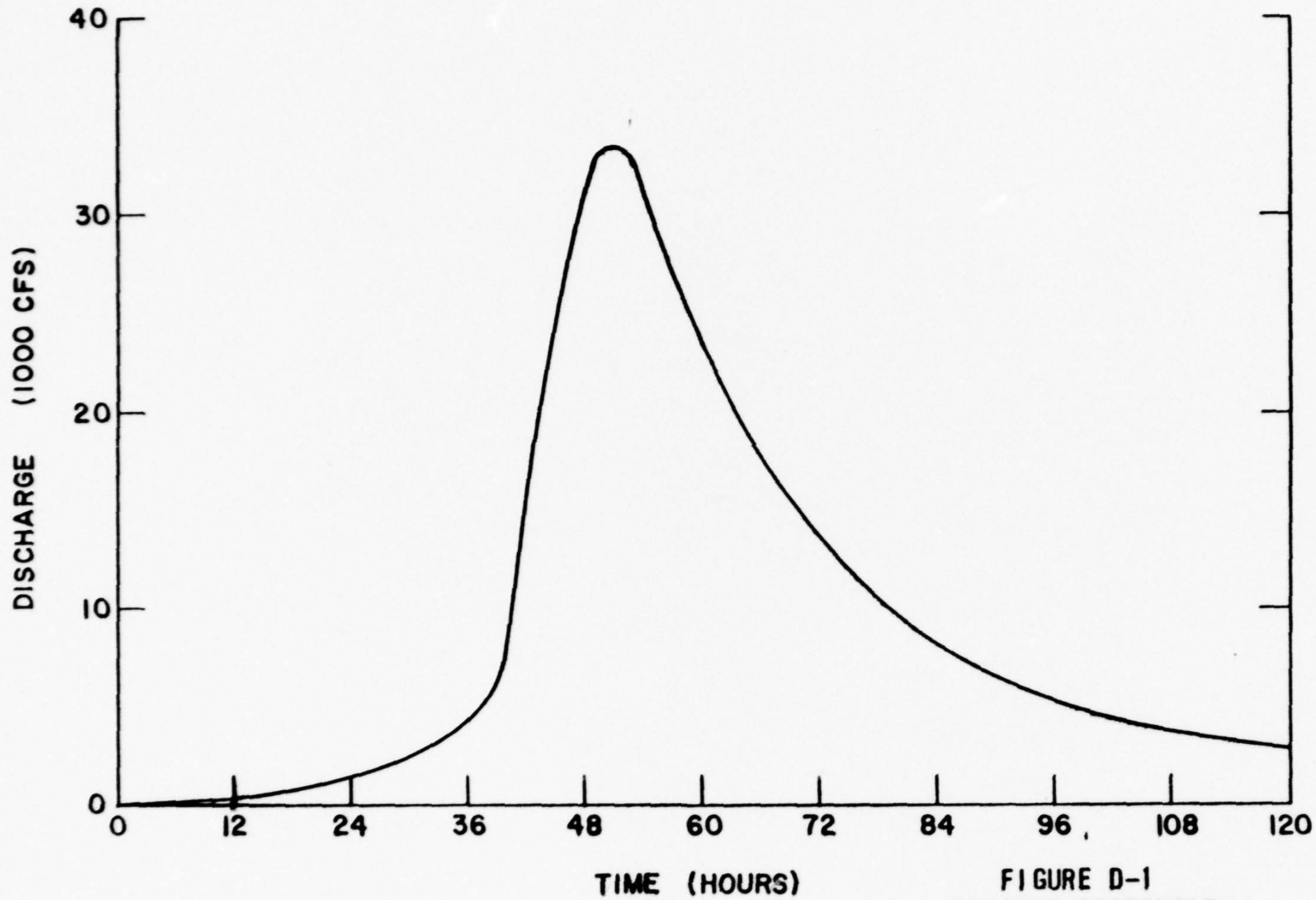


FIGURE D-1
WANAQUE RESERVOIR
PROBABLE MAXIMUM FLOOD
INFLOW HYDROGRAPH

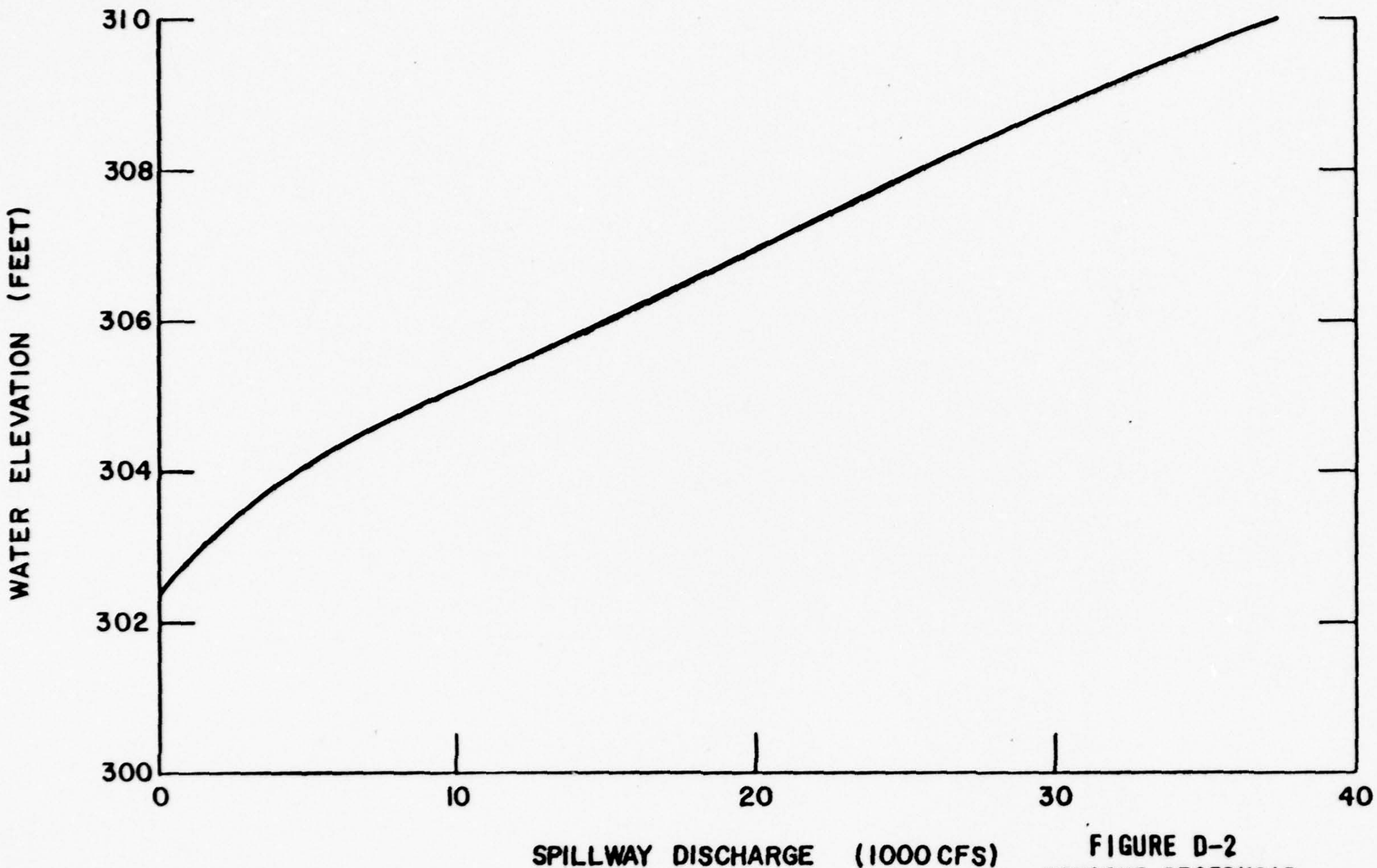


FIGURE D-2
WANAQUE RESERVOIR
OVERFLOW SPILLWAY
DISCHARGE RATING CURVE

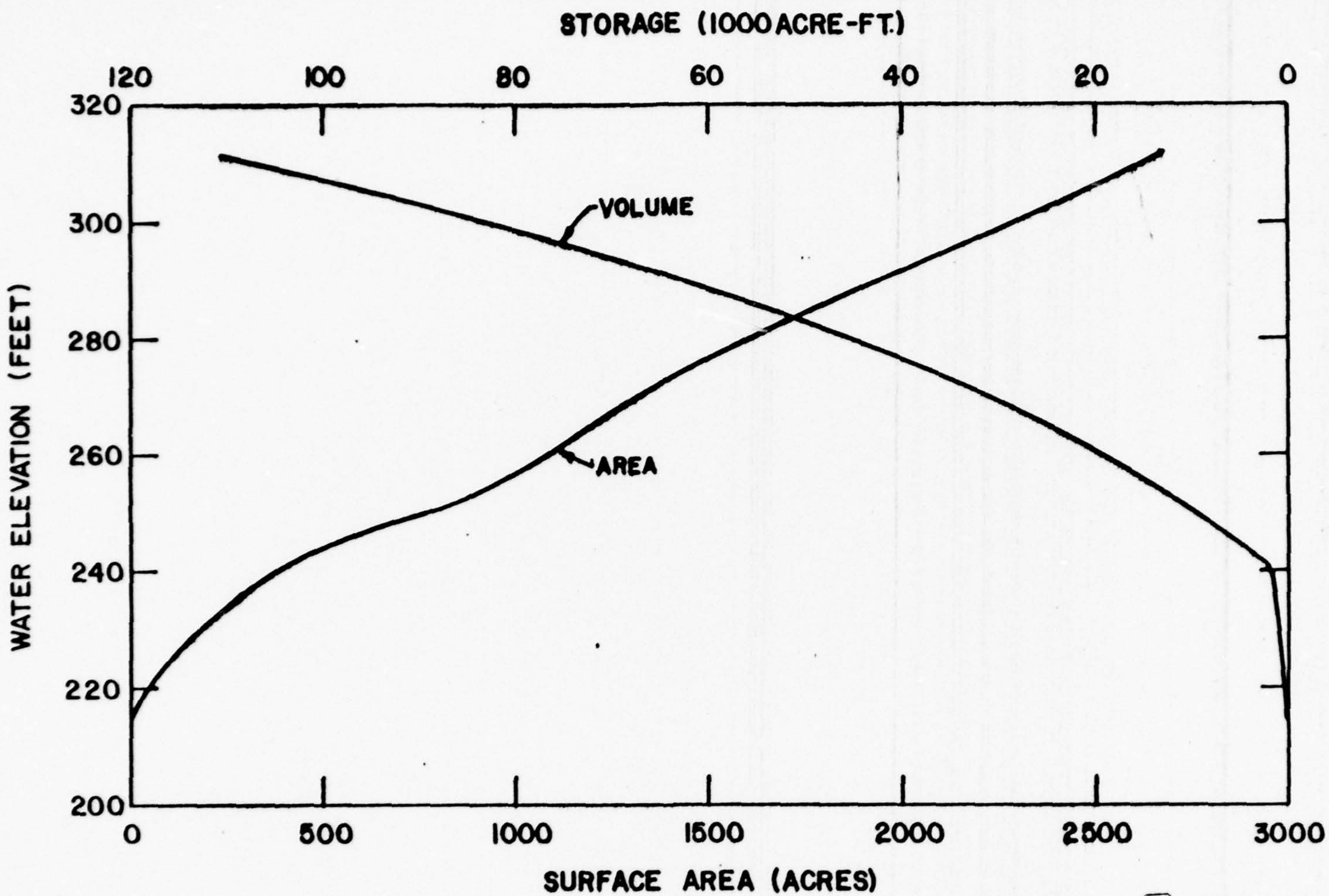


FIGURE D-3
WANAQUE RESERVOIR
AREA - CAPACITY CURVES

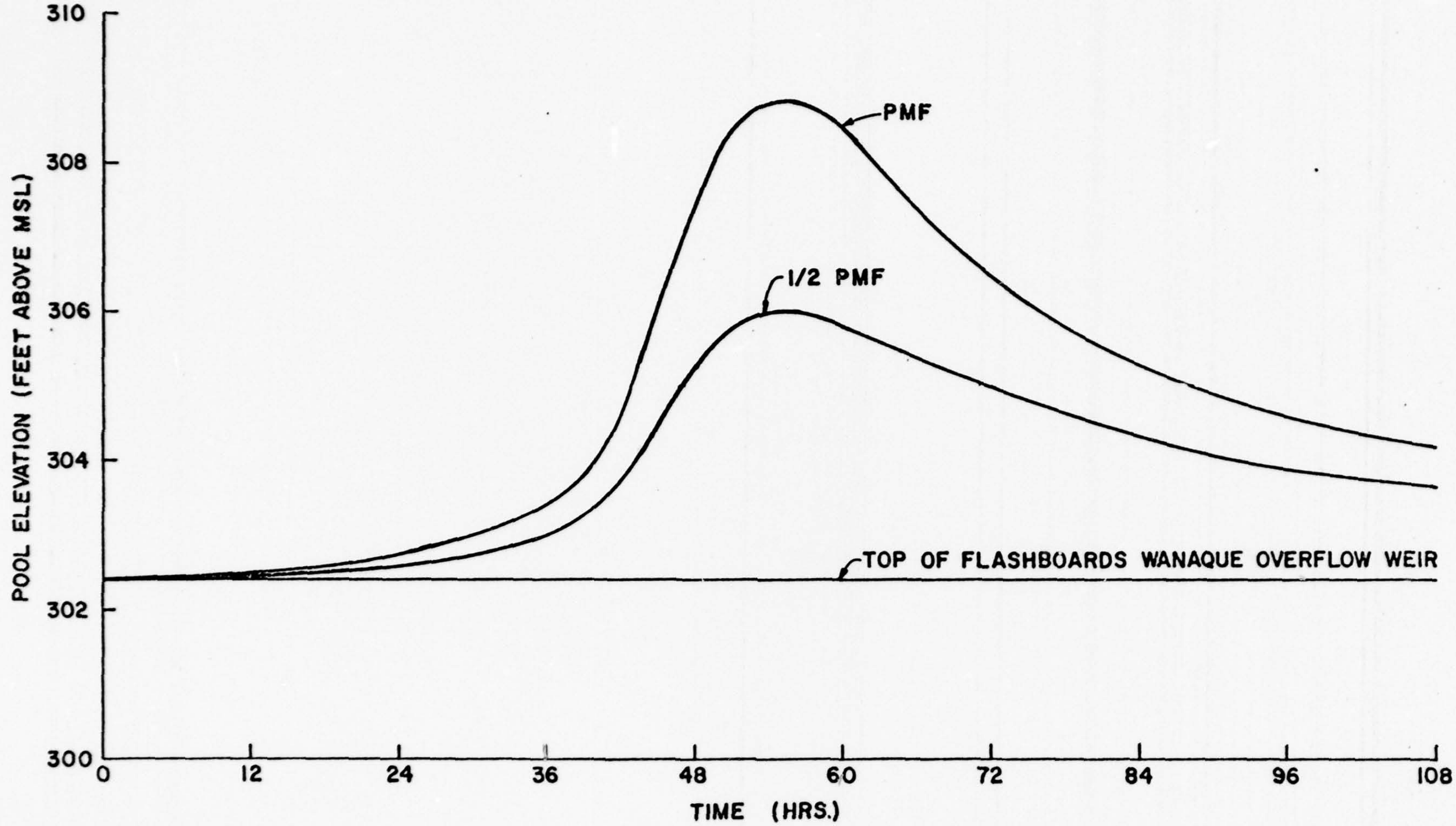


FIGURE D-4
FLOOD ROUTING THROUGH
WANAQUE RESERVOIR WITH
FLASHBOARDS IN PLACE

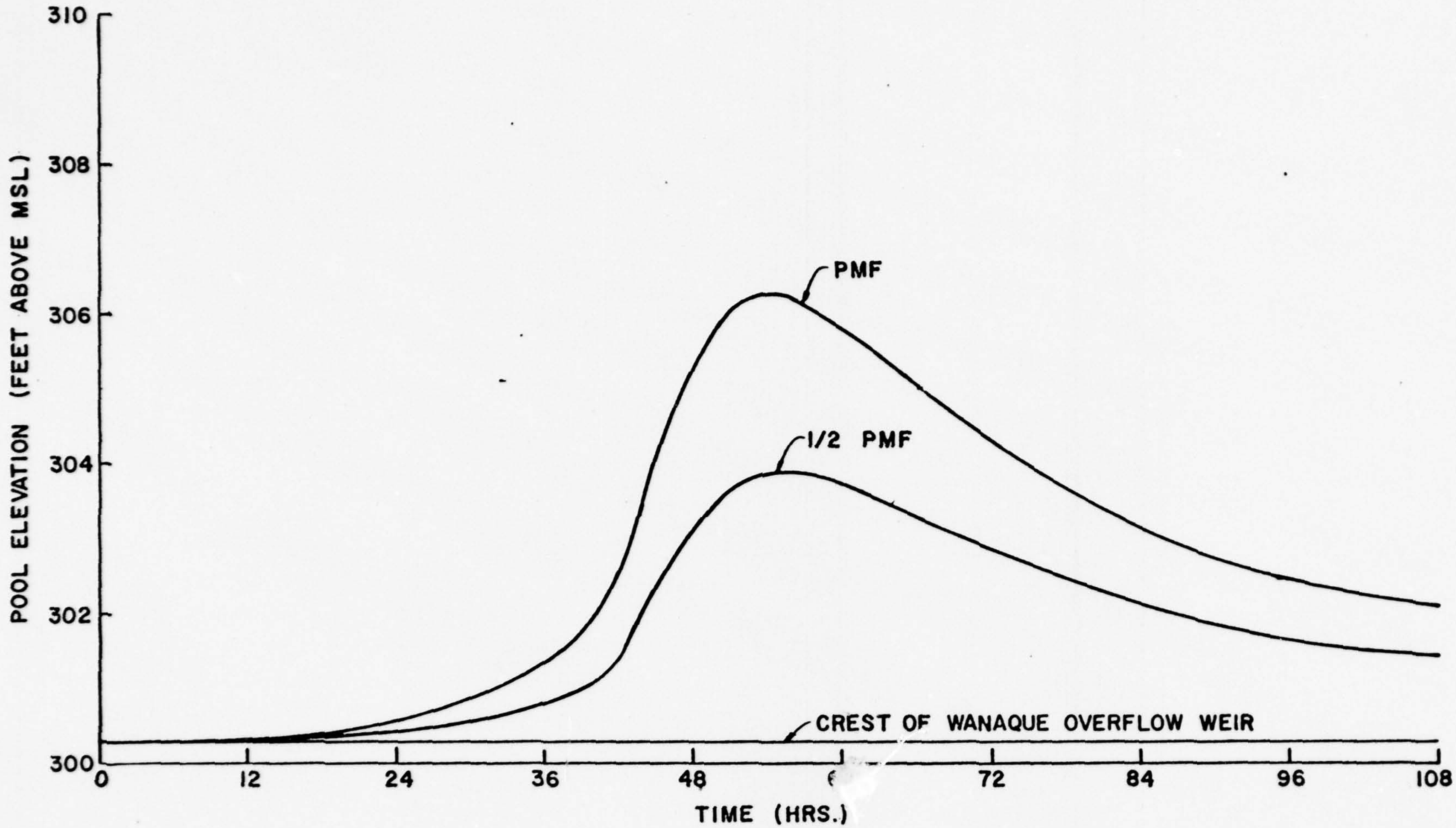


FIGURE D-5
FLOOD ROUTING THROUGH
WANAQUE RESERVOIR <u>WITHOUT</u>
FLASHBOARDS IN PLACE

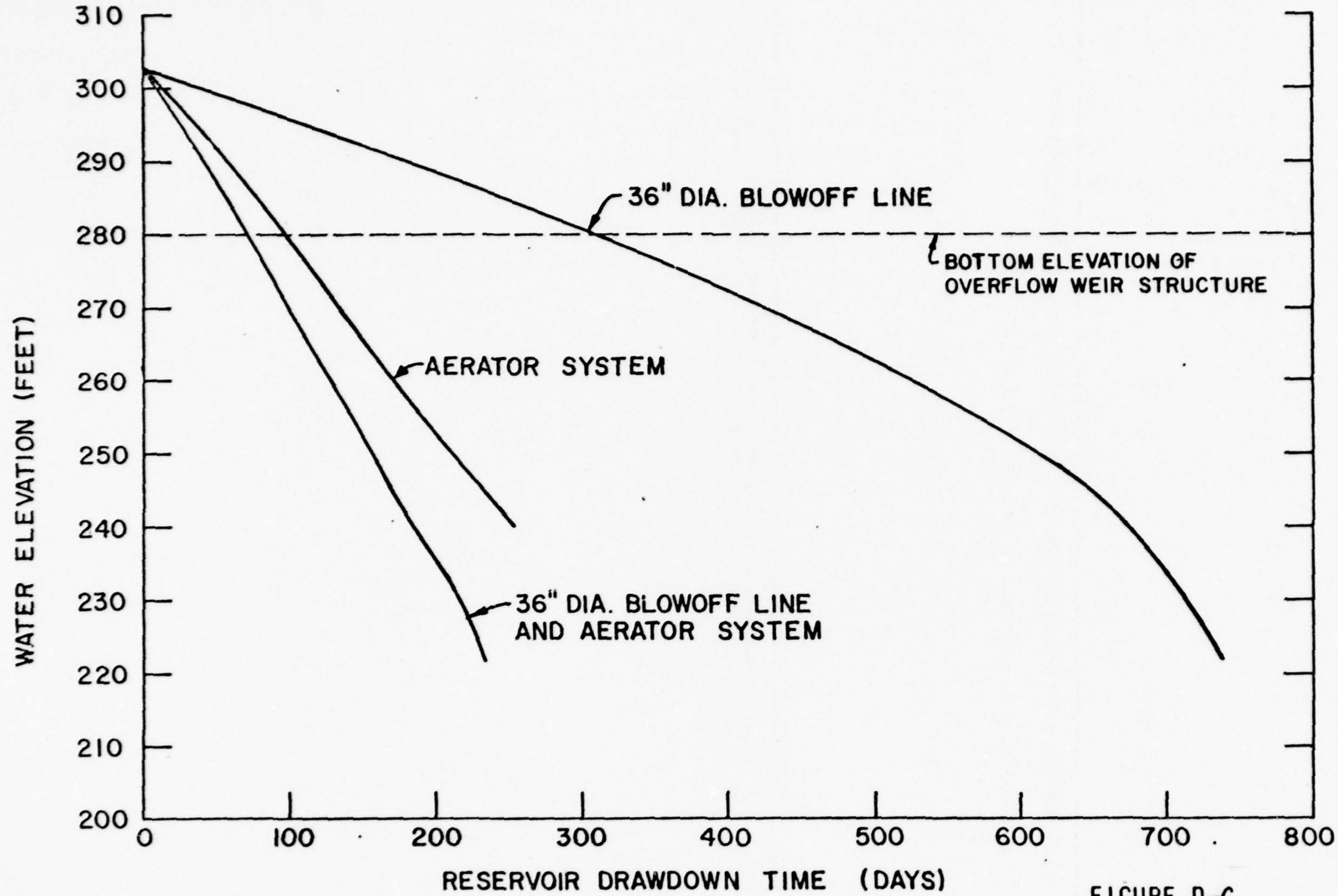


FIGURE D-6
WANAQUE RESERVOIR
TIME OF DRAWDOWN

CALCULATION FOR

$L_{18"}$ = 32'

$L_{36"}$ = 200'

ENTRANCE

REDUCTION

24" VALVE

EXPANSION

45° ELBOW

REDUCTION

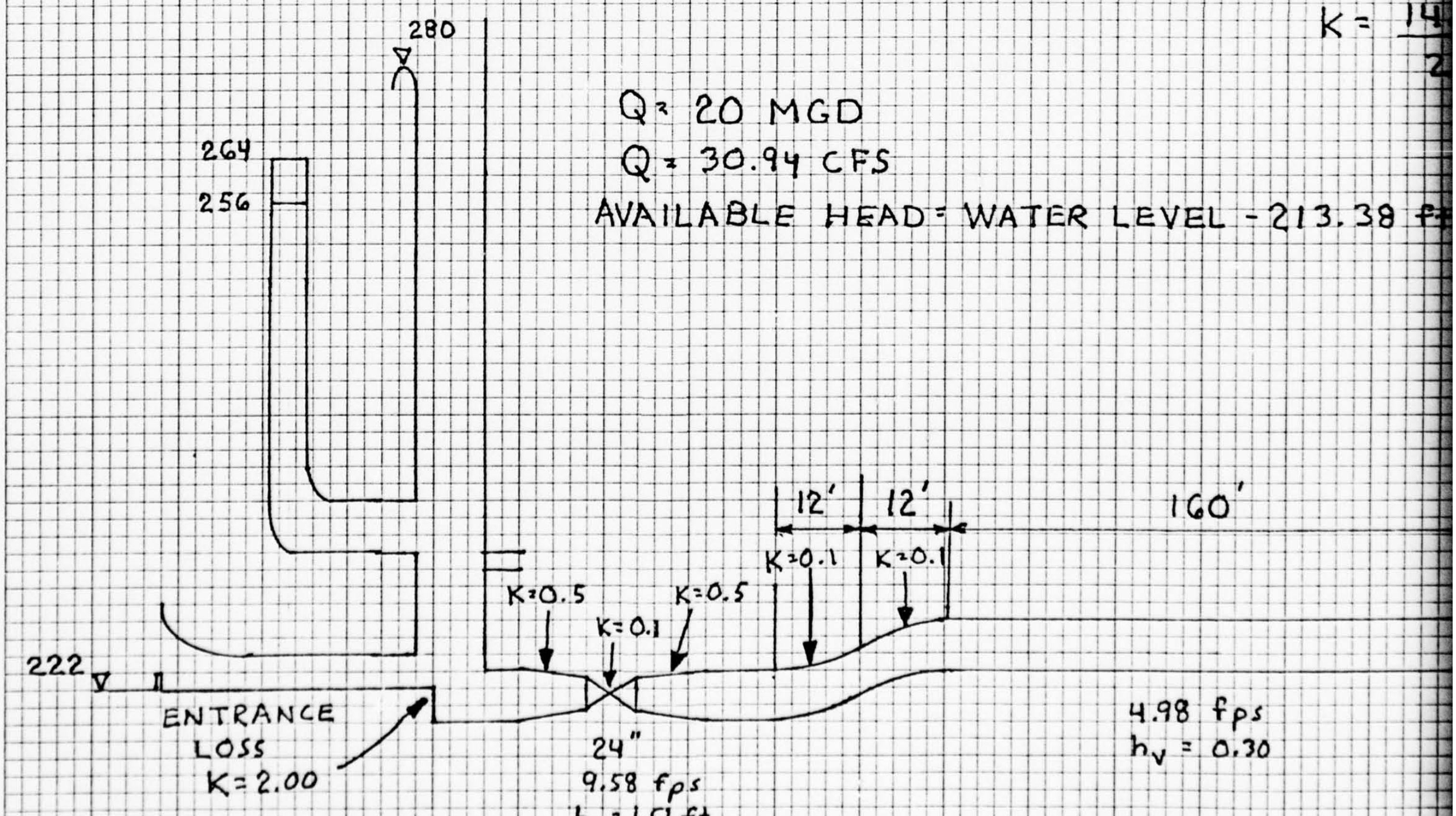
DISCHARGE

TOTAL LOSS

EQUATION FO

$\Delta h = k\,(Q$

$K = \frac{14}{2}$



ULATION FOR 36" Ø BLOWOFF DRAWDOWN

$L_{18"} = 32'$ $\Delta h = \frac{113}{1000} \times 32 = 3.62'$

$L_{36"} = 200'$ $\Delta h = \frac{3.86}{1000} \times 200 = 0.77'$

ENTRANCE $K = 2.00$; $\Delta h = 0.30 \times 2.0 = 0.60'$

REDUCTION $K = 0.50$; $\Delta h = 0.5 \times 1.51 = 0.76'$

24" VALVE $K = 0.10$; $\Delta h = 0.1 \times 1.51 = 0.15'$

EXPANSION $K = 0.50$; $\Delta h = 0.5 \times 1.51 = 0.76'$

45° ELBOW $K = 0.10$; $\Delta h = 0.1 \times 0.3 \times 2 = 0.06'$

REDUCTION $K = 0.50$; $\Delta h = 0.5 \times 4.75 = 2.83'$

DISCHARGE $K = 1.00$; $\Delta h = 1.0 \times 4.75 = 4.75'$

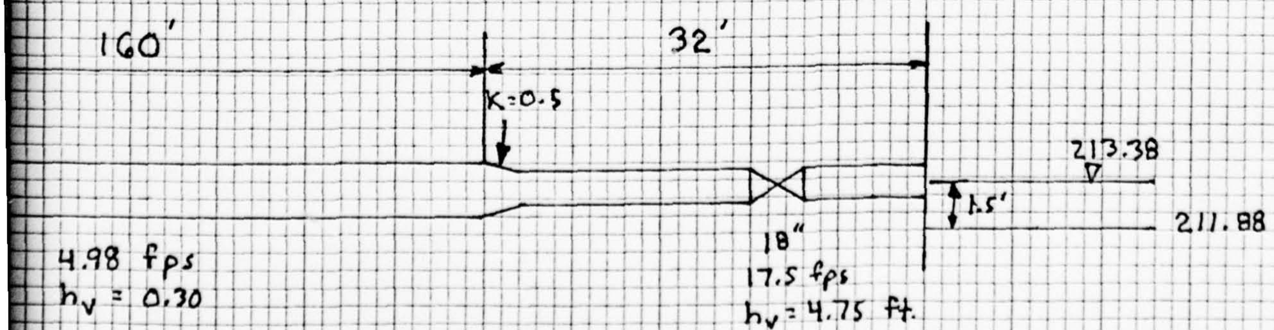
TOTAL LOSS FOR 20 MGD FLOW 14.30'

EQUATION FOR HEAD LOSS and MGD IS:

$$\Delta h = k\,(Q_{MGD})^2 \Rightarrow Q \sqrt{\frac{\Delta h}{k}}$$

$$K = \frac{14.30}{20^2} = \frac{14.30}{400} = 0.03575$$

LEVEL - 213.38 ft.



2

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	CLIENT Corps. of Eng.	FILING CODE
	PROJECT Dam Inspection	W.O. / PAGE 6 OF
SYSTEM Wanaque Reservoir		ORIGINATOR Wahomite PRR DATE July 19/79
CALCULATION FOR 36" Ø Blowoff Drawdown		REVIEWER R.A. Nott PRR DATE 7-20-79

RESULTS

2 cfs / sq. mi.

2 × 90.4 = 180.8 cfs inflow = 117 MGD

Average consumption

117 − 100 = +17 MGD

Assuming that inflow + Daily consumption balance out the 36" bypass can outflow.

Center of Gravity of zone considered	Storage Million Gallons	Available head differential feet	Q MGD	Partial time for Zone Days	Reservoir Water level
226	280	12.62	18.79	14.90	
					230
235	900	21.62	24.59	36.60	
					240
245	1900	31.62	29.74	63.89	
					250
255	3000	41.62	34.12	87.92	
					260
265	4000	51.62	38.00	105.27	
					270
275	4600	61.62	41.52	110.80	
					280
285	5900	71.62	44.76	131.82	
					290
295	6900	81.62	47.78	144.41	
					300
305	7000	91.62	50.62	138.27	
					310

These numbers have been calculated assuming that the minimum inflow of 2 cfs/sq. mi. concentrates in the water consumption downstream.

FILING CODE

36 ϕ Blowoff Drawdown

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	CLIENT Corps. of Eng.	FILING CODE
	PROJECT Dam Inspection	W.O. / PAGE 7 OF
SYSTEM Wanaque Reservoir		ORIGINATOR Wahanik PRR DATE July 19/78
CALCULATION FOR 36" ϕ Blowdown Drawdown		REVIEWER R.A.RH PRR DATE 7-20-78

DRAWDOWN Calcs
Using the ~~drawdown~~ 36" Blowoff
for water below EL 302.4'

time required between 310 and 300 is 138.27 Days
therefore:

a) between 300 and 302.4 = $\frac{138.27}{10} \times 2.4 = 33.18$ Days.

Reservoir water level	Partial time Days	Accumulated total time DAYS
302.4		0
	33.18	
300		33.18
	144.41	
290		177.59
	131.82	
280		309.41
	110.80	
270		420.21
	105.27	
260		525.48
	87.92	
250		613.40
	63.89	
240		677.29
	36.60	
230		713.89
222*	14.90	728.79

RESULTS

* lowest feasible water level - entrance lower tunnel.

GAI 350 REV. 10-72

CALCUL

THE BASIC DATA FOR FRICTION LOSS CALCULATIONS IS:

Ø	HEAD LOSS IN 1000 FEET	FLOW MGD	FLOW CFS	VELOCITY FPS	$\frac{V^2}{2g}$
36	17.4	45	69.6	9.85	1.50
24	16.3	15	23.2	7.39	0.85
48	7.3	60	92.8	7.39	0.85
72	3.10	110	170.2	6.02	0.56
144	0.381	220	340	3.01	0.14

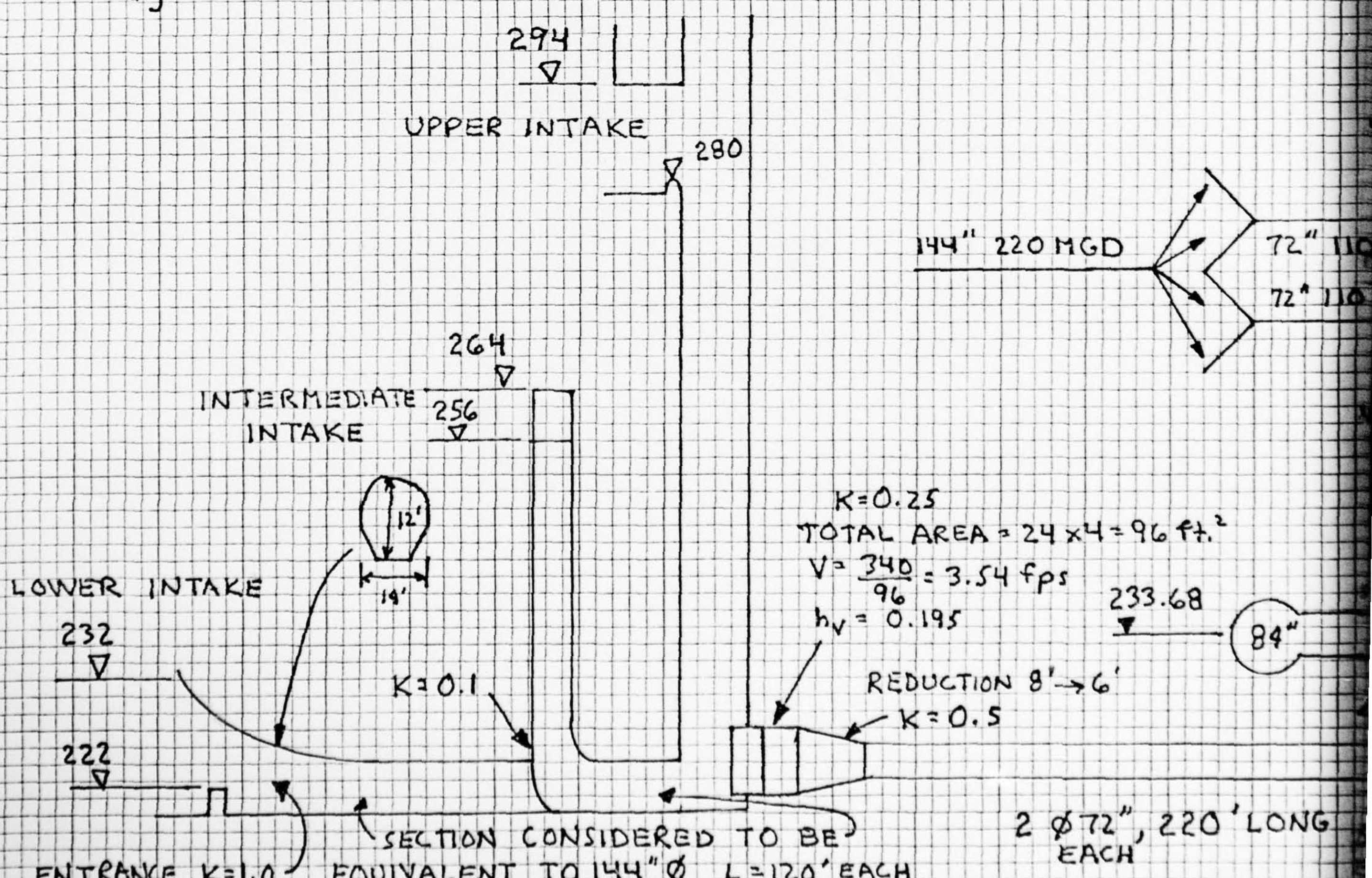
A_{33} Ø $1\frac{3}{8}$" NOZZLE = 33×0.0103

A_{133} Ø 5" NOZZLE = 133×0.0341

$A_{33} + A_{133} = 0.34 + 4.50 = 4.84$ sq. ft.

$V_{water} = \frac{185.7}{4.84} = 38.37$ fps

$\frac{V^2}{2g} = 22.9$

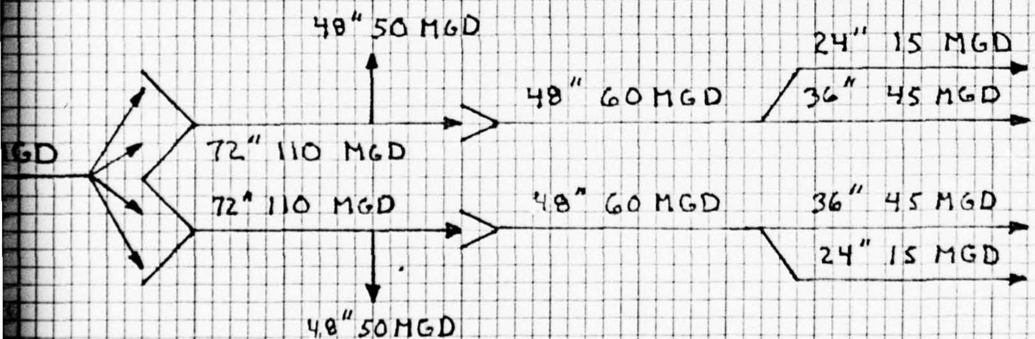


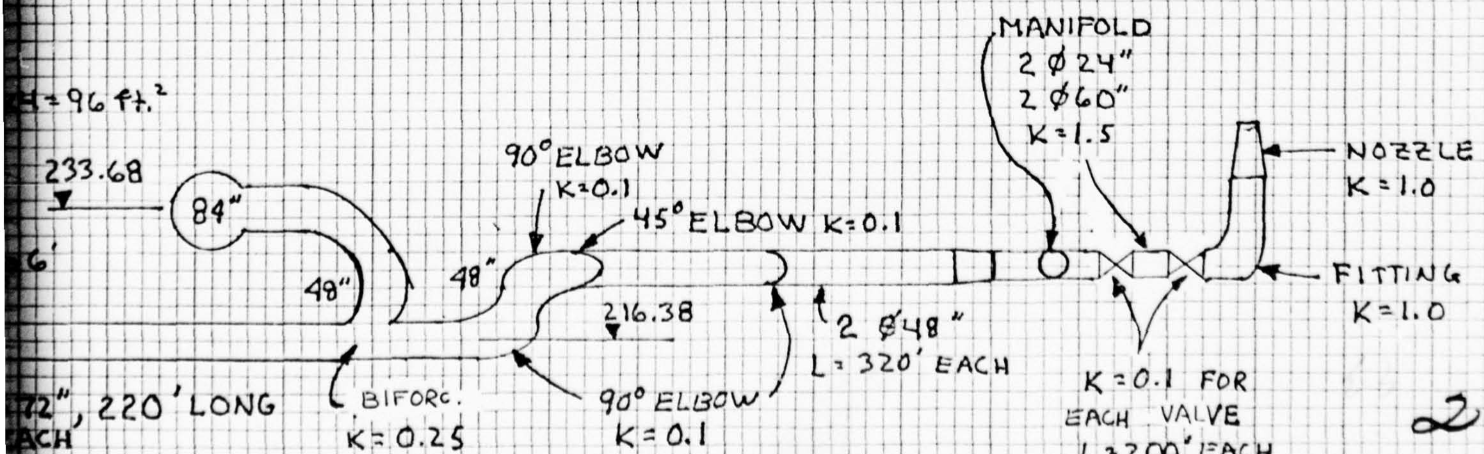
1

CALCULATION FOR AERATOR DRAWDOWN

S:
.50
.85
.85
.56
.14

LOSS	K	Q MGD	$\frac{V^2}{2g}$	HEAD LOSS IN 1000 FEET	L Feet	HEAD LOSSES
ENTRANCE	1.00	220	0.14			0.140
FRICTION 144"Ø PIPE		220		0.381	120	0.046
GATES	0.25	220	0.195			0.049
REDUCTION	0.50	110	0.56			0.280
FRICTION 72"Ø PIPE		110		3.10	220	0.682
BIFORC.	0.25	110	0.56			0.140
90° ELBOW	0.10	60	0.85			0.085
90° ELBOW	0.10	60	0.85			0.085
45° ELBOW	0.05	60	0.85			0.043
FRICTION 48"Ø PIPE		60		7.30	320	2.336
90° ELBOW	0.10	60	0.85			0.085
MANIFOLD & REDUCERS	1.50	60	0.85			1.275
FRICTION 36"Ø PIPE		45		17.4	200	3.480
FITTINGS & VALVES	1.20	45	1.50			1.800
NOZZLES	1.0	120	22.90			22.900
				TOTAL LOSSES	=	33.426





Aerator Drawdown.

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	CLIENT Corps of Eng.	FILING CODE
	PROJECT DAM INSPECTION	W.O. PAGE 9 OF
SYSTEM WANA QUE RESERVOIR		ORIGINATOR Wahanik DATE July 19/78
CALCULATION FOR AERATOR DRAWDOWN		REVIEWER R. A. Pott DATE 7-20-78

RESULTS

the aerator system can be used to draw water down to elevation 240.5'. In all instances is being assumed that inflow to the reservoir equal water consumption.

the head losses equation is

$$\Delta h = \left(Q_{MGD}\right)^2 * K$$

$$K = \frac{33.426}{(120)^2} = 0.00232125$$

and the times required for drawdown are a function of $Q_{MGD} = \left(\frac{\Delta h}{K}\right)^{1/2}$ and the partial storage.

FILING CODE

GAI 350 REV. 10-72

Aerator Drawdown.

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	CLIENT: CORPS of ENG.	FILING CODE
	PROJECT: DAM INSPECTION	W.O. — PAGE 10 OF
SYSTEM: WANAQUE RESERVOIR		ORIGINATOR: Wahanik PRR DATE: July 19/78
CALCULATION FOR: AERATOR DRAWDOWN		REVIEWER: R. A. Putt PRR DATE: 7-20-78

RESULTS

Center of Gravity of Zone considered.	Partial Storage Million Gallons.	Reservoir water level ft.	Available Head diff. above top of Nozzle that is at elevation 240.5 ft	Q MGD	Partial time to empty layer of 10' depth DAYS	total Accumulated time to empty Reservoir with Aerator System DAYS
		240.5				253.65
245	1805		4.50	44.03	41.00	
		250				212.65
255	3000		14.50	79.09	37.96	
		260				174.69
265	4000		24.5	102.74	38.93	
		270				135.76
275	4600		34.5	121.91	37.73	
		280				98.63
285	5900		44.5	138.46	42.61	
		290				55.42
295	6400		54.5	153.23	45.03	
		300				10.39
301.2	1680		60.7	161.71	10.39	
		302.4				0

FILING CODE

Combined Drawdown

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	CLIENT Corps. of Eng.	FILING CODE
	PROJECT DAM INSPECTION	W.O. / PAGE 11 OF
SYSTEM WANAQUE RESERVOIR		ORIGINATOR Wahanik PRR DATE July 19/78
CALCULATION FOR COMBINED DRAWDOWN		REVIEWER R.A. Pott PRR DATE 7-20-78

RESULTS

the aerator working together with The 36" Blowoff could draw the reservoir much faster the data is

Center of Gravity of Zone Considered ft.	Reservoir water level	Partial Storage in Million Gallons	Flow 36" Blowoff MGD	Flow aerator MGD	total combined flow MGD	Partial time to empty layer Considered DAYS	Total Accumulated Days needed to lower Reservoir
	240.5						181.98
245		1805	29.86	44.03	73.89	24.43	
	250						157.55
255		3000	34.12	79.09	113.21	26.50	
	260						131.05
265		4000	38.00	102.74	140.74	28.42	
	270						102.63
275		4600	41.52	121.91	163.43	28.15	
	280						74.48
285		5900	44.76	138.46	183.22	32.20	
	290						42.28
295		6900	47.78	153.23	201.01	34.33	
	300						7.95
301.2		1680	49.56	161.71	211.27	7.95	
	302.4						0

FILING CODE

GAI 350 REV. 10-72

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	CLIENT: Corps. of Eng.	FILING CODE
	PROJECT: DAM INSPECTION	W.O. / PAGE 12 OF
SYSTEM: WANAQUE RESERVOIR		ORIGINATOR: Wahanik PRR DATE July 19/78
CALCULATION FOR: DRAWDOWN DATA		REVIEWER: R.A. Roth PRR DATE 7-20-78

RESULTS

Reservoir draw down Data

IN DAYS of 24 hours.

Reservoir Water level ft.	Overflow Weir	36 inch diameter Blowoff At EL 213.38	Aerator in operation with nozzles at elevation 240.5 ft.	Combined operation of 36" Ø Blowoff and Aerator in operation.
		0	0	0
300		33.18	10.39	7.95
290		177.59	55.42	42.28
280		309.41	98.03	74.48
270		420.21	135.76	102.63
260		525.48	174.69	131.05
250		613.40	212.65	157.55
240		677.29	253.65	181.98
230		713.89		218.58 *
222		728.79		233.48 *

* Below el. 240.5 ft only the 36" Ø Blowoff is in operation.

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	CLIENT: COE	FILING CODE
	PROJECT: NJ DAM INSPECTIONS	W.O. / PAGE 1 OF
SYSTEM: WANAQUE RESERVOIR		ORIGINATOR: R.A. PUTT DATE 5-18-78
CALCULATION FOR: DISCHARGE OF OVERFLOW WEIR		REVIEWER: [illegible] DATE 7/13/78

RESULTS

I) OVERFLOW WEIR IS AN UNCONTROLLED

OGEE CREST SPILLWAY.

ASSUME FLASHBOARDS ARE REMOVED.

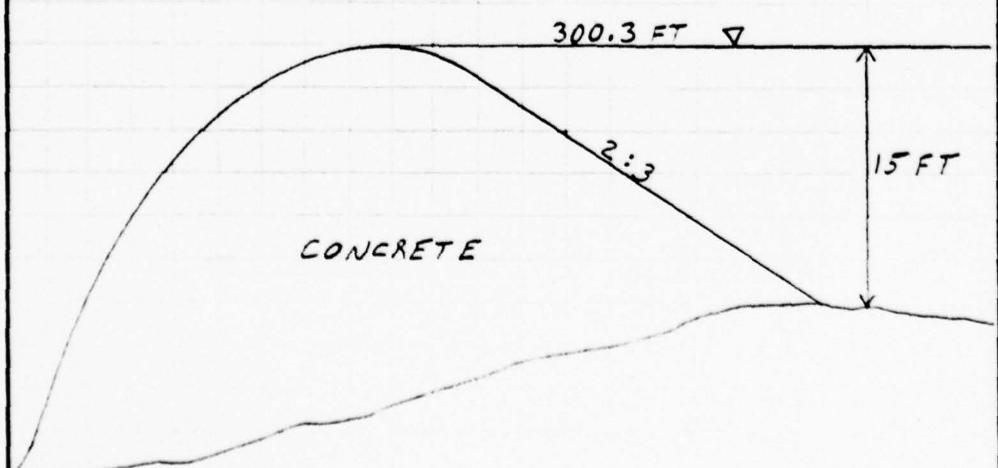
NET WEIR LENGTH = 520 FT.

NUMBER OF PIERS = 39

P = UPSTREAM DEPTH = 15 FT

UPSTREAM SLOPE = 2:3

CREST ELEVATION = 300.3 FT



GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	CLIENT: COE	FILING CODE
	PROJECT: NJ DAM INSPECTIONS	W.O. / PAGE 2 OF
SYSTEM: WANAQUE RESERVOIR		ORIGINATOR R.A.P. DATE 7-12-78
CALCULATION FOR: DISCHARGE OF OVERFLOW WEIR		REVIEWER JDF DATE 7/19/78

RESULTS

ASSUME NEGLIGIBLE VELOCITY HEAD BECAUSE $1.33\,H < 15$ FT, ALWAYS

A) APPROACH HEAD = 0.0 FT

$Q = 0.0$ CFS

B) APPROACH HEAD = 0.5 FT

$P/H = 15/0.5 = 30$

$C_V = C_{VERTICAL} = 3.95$

$C_I = C_{INCLINED} = C_V = 3.95$

L = EFFECTIVE LENGTH

$= 520 - 2(39 \times 0.02 + 0.2)0.5 = 519$ FT

ASSUMED SQUARE PIERS AND ABUTMENTS

$Q = CLH^{3/2} = (3.95)(519)(0.5)^{3/2} = 725$ CFS ✓

C) APPROACH HEAD = 1.0 FT

$P/H = 15 \qquad C = 3.95$

$L = 520 - 1.96 = 518$ ✓

$Q = (3.95)(518)(1.0)^{3/2} = 2046$ ✓

FILING CODE

GAI 350 REV. 10-72

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	CLIENT COE	FILING CODE
	PROJECT NJ DAM INSPECTIONS	W.O. / PAGE 3 OF
SYSTEM WANAQUE RESERVOIR		ORIGINATOR R. A. [illegible] DATE 5-17-78
CALCULATION FOR DISCHARGE OF OVERFLOW WEIR		REVIEWER [illegible] DATE 7/18/78

RESULTS

SINCE P/H WILL ALWAYS BE ABOVE 1.5,

C_I WILL ALWAYS BE EQUAL TO C_V.

TABLE OF DISCHARGE DEVELOPMENT -

APPROACH HEAD (FT)	P/H	C	EFFECTIVE LENGTH (FT)	DISCHARGE (CFS)
0.0	–	–	–	0
0.5	30	3.95	519	725
1.0	15	3.95	518	2050
1.5	10	3.95	517	3750
2.0	7.5	3.95	516	5760
2.5	6	3.95	515	8040
3.0	5	3.95	514	10550
3.5	4.29	3.95	513	13270
4.0	3.75	3.95	512	16180
4.5	3.33	3.95	511	19270
5.0	3.00	3.95	510	22520
5.5	2.73	3.95	509	25930
5.7 *	2.63	3.94	509	27290
6.0 **	2.50	3.94	508	29420

* TOP OF ABUTMENT
** BOTTOM OF BRIDGE

REFERENCE : DESIGN OF SMALL DAMS

FILING CODE

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	CLIENT COE	FILING CODE	
	PROJECT NJ DAM INSPECTIONS	W.O.	PAGE 1 OF
SYSTEM WANAQUE RESERVOIR		ORIGINATOR R A PUTT	DATE 5-13-79
CALCULATION FOR DISCHARGE OF OVERFLOW WEIR		REVIEWER WF	DATE 7/18/79

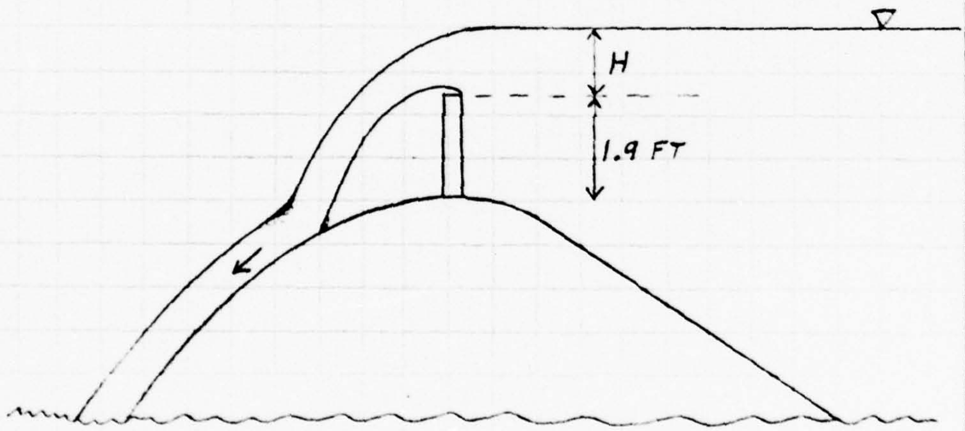
RESULTS

II) ASSUME FLASHBOARDS REMAIN INTACT.

FLASHBOARDS WILL ACT AS SHARP-CRESTED WEIR.

ELEVATION OF TOP OF FLASHBOARDS = 302.2 FT

FLASHBOARD HEIGHT = 1.9 FT



DISCHARGE EQUATION:

$Q = CLH^{3/2}$, WHERE:

$C = 3.27 + 0.40\frac{H}{h}$, $h = 1.9$ FT

EFFECTIVE LENGTH DETERMINED BY

$L = 520 - 2(39 \times 0.02 + 0.2)H = 520 - 1.96H$

GAI 350 REV. 10-72

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	CLIENT COE	FILING CODE
	PROJECT NJ DAM INSPECTIONS	W.O. PAGE 5 OF
SYSTEM WANAQUE RESERVOIR		ORIGINATOR R A PUTT DATE 5-11-78
CALCULATION FOR DISCHARGE OF OVERFLOW WEIR		REVIEWER DATE 7/13/78

RESULTS

TABLE OF CALCULATED DISCHARGES

APPROACH HEAD (FT)	C	EFFECTIVE LENGTH (FT)	DISCHARGE (CFS)
0.0	-	-	0
0.5	3.38	519	620
1.0	3.48	518	1800
1.5	3.59	517	3410
2.0	3.69	516	5390
2.5	3.80	515	7740
3.0	3.90	514	10420
3.5	4.01	513	13470
3.8 *	4.07	513	15470
4.0 **	4.12	512	16880
4.5 ***	4.22	511	20590
5.0 ***	4.32	510	24630
5.5 ***	4.43	509	29080

* TOP OF ABUTMENTS
** BOTTOM OF BRIDGE
*** ASSUME BRIDGE FAILS

REFERENCE: CHOW, OPEN-CHANNEL HYDRAULICS

FILING CODE

GAI 350 REV. 10-72

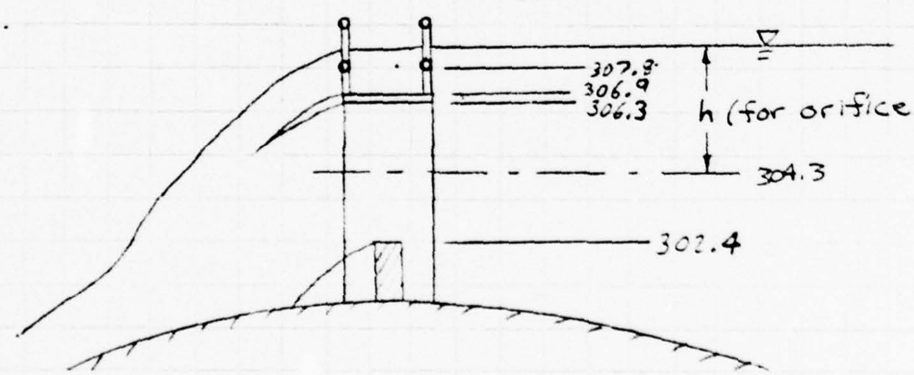
GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	CLIENT COE	FILING CODE
	PROJECT NJ DAM INSPECTIONS	W.O. / PAGE OF
SYSTEM WANAQUE RESERVOIR		ORIGINATOR R A POTT
CALCULATION FOR ELEVATION - DISCHARGE		DATE 6-2-78
		REVIEWER
		DATE

RESULTS

ASSUME FLASHBOARDS REMAIN IN PLACE AND WALKWAY BRIDGE REMAINS INTACT.

BELOW ELEVATION 306.3, THE SPILLWAY WITH FLASHBOARDS ACTS AS A SHARP-CRESTED WEIR.

ABOVE ELEVATION 306.3, THE FLOW UNDER THE BRIDGE IS ORIFICE FLOW AND ABOVE THE BRIDGE IS SHARP-CRESTED WEIR FLOW.



GAI 350 REV. 10-72

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	CLIENT COE	FILING CODE
	PROJECT N J DAM INSPECTIONS	W.O. PAGE 7 OF
SYSTEM WANAQUE RESERVOIR		ORIGINATOR R A SOTT
		DATE
CALCULATION FOR ELEVATION - DISCHARGE		REVIEWER
		DATE
		RESULTS

WATER SURFACE ELEVATION AT BOTTOM OF BRIDGE = 306.2 FT

Q = 16880 CFS FROM WEIR EQUATION ✓

ORIFICE EQUATION:

$Q = CA\sqrt{2gH}$

$Q = 15470$ CFS
$A = 520 \times 3.8 = 1976$ FT^2
$H = 1.9$ FT

$C = 15470 / 1976\sqrt{3.8g} = 0.71$ ✓

AT ELEVATION 306.9 - NO FLOW OVER BRIDGE, ONLY ORIFICE FLOW UNDER:

$Q = (0.71)(1976)\sqrt{2g(2.6)} = 18150$ CFS

AT ELEVATION 307.4 -

$Q_{UNDER} = (0.71)(1976)\sqrt{2g(3.1)} = 19800$ CFS

$Q_{OVER} = CLH^{3/2}$

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	CLIENT: COE	FILING CODE
	PROJECT: N J DAM INSPECTIONS	W.O. / PAGE OF
SYSTEM: WANAQUE RESERVOIR		ORIGINATOR: R A PUTT / DATE
CALCULATION FOR: ELEVATION - DISCHARGE		REVIEWER / DATE 7/10/78

RESULTS

BECAUSE OF BRIDGE WIDTH AND 90 VERTICAL RAILINGS ON BOTH SIDES, ASSUME A LOW CONSTANT C VALUE = SHARP-CRESTED MINIMUM = 3.27

$$L = 551.7 - 80\left(\frac{1}{6}\right) = 538 \text{ FT}$$

$$Q_{OVER} = (3.27)(538)(0.5)^{1.5} = 600 \text{ CFS}$$

$$Q_{TOTAL} = 19800 + 600 = 20400 \text{ CFS}$$

AT ELEVATION 307.8 -

$$Q_{UNDER} = (0.71)(1976)\sqrt{2g(3.5)} = 21100 \text{ CFS}$$

$$Q_{OVER} = (3.27)(538)(0.9)^{1.5} = 1500 \text{ CFS}$$

$$Q_{TOTAL} = \overset{21100}{\cancel{22500}} + 1500 = 22600 \text{ CFS}$$

A 2-INCH DIAMETER HORIZONTAL PIPE RAILING EXISTS AT 307.8 FT, THE FLOW MUST BE ADJUSTED BECAUSE OF THIS BLOCKAGE.

FILING CODE

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	CLIENT COE	FILING CODE
	PROJECT NJ DAM INSPECTIONS	W.O. / PAGE 9 OF
SYSTEM WANAQUE RESERVOIR		ORIGINATOR
		DATE
CALCULATION FOR		REVIEWER DJF
		DATE 7/18/78
		RESULTS

AT ELEVATION 308.5 FT -

$Q_{UNDER} = (0.71)(1976)\sqrt{2g(4.2)} = 23100$ CFS

$Q_{OVER} = (3.27)(538)(1.6)^{1.5}\left(1 - \frac{2}{19}\right) = 3200$ CFS

$Q_{TOTAL} = \overset{23100}{\cancel{24600}} + 3200 = 26300$ CFS

AT ELEVATION 309.0 FT -

$Q_{UNDER} = (0.71)(1976)\sqrt{2g(4.7)} = 24400$ CFS

$Q_{OVER} = (3.27)(538)(2.1)^{1.5}\left(1 - \frac{2}{25}\right) = 4900$ CFS

$Q_{TOTAL} = \overset{24400}{\cancel{26000}} + 4900 = 29300$ CFS

FILING CODE

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	CLIENT COE	FILING CODE	
	PROJECT N. J. DAM INSPECTIONS	W.O.	PAGE 1 OF
SYSTEM WANAQUE RESERVOIR		ORIGINATOR R. A. PUTT DATE 5-13-78	
CALCULATION FOR RESERVOIR AREA		REVIEWER DATE 7/19/78	

RESULTS

A) AT ELEVATION 300.0 FT

AREA = 25.00 PLANIMETER UNITS

$= 100,000,000 \ FT^2$

= 2296 ACRES

B) AT ELEVATION 320.0 FT

AREA = 31.80 PLANIMETER UNITS

$= 127,200,000 \ FT^2$

= 2920 ACRES

FILING CODE

GAI 350 REV. 10-72

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	CLIENT: C/E	FILING CODE
	PROJECT: NJ DAM INSPECTIONS	W.O. / PAGE 11 OF
SYSTEM: WANAQUE RESERVOIR		ORIGINATOR: R A PITT / DATE 5-11-78
CALCULATION FOR: VOLUME - DISCHARGE RELATION		REVIEWER / DATE 5/10/78

RESULTS

WITH FLASHBOARDS REMOVED

POOL ELEVATION (FT ABOVE MSL)	SURFACE AREA (ACRES)	STORAGE VOLUME (AC-FT)	DISCHARGE (CFS)
300.3	2305	0	0
300.8	2321	1160	725
301.3	2337	2320	2050
301.8	2352	3490	3750
302.3	2368	4670	5760
302.8	2383	5860	8040
303.3	2399	7060	10550
303.8	2415	8260	13270
304.3	2430	9470	16130
304.8	2446	10690	19270
305.3	2461	11920	22520
305.8	2477	13150	25930
306.0	2483	13650	27290
306.2	2490	14150	28680

FILING CODE

GAI 350 REV. 10-72

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	CLIENT COE	FILING CODE
	PROJECT NJ DAM INSPECTIONS	W.O. / PAGE 12 OF

SYSTEM WANAQUE RESERVOIR

ORIGINATOR

DATE 5-2-78

CALCULATION FOR VOLUME - DISCHARGE RELATION

REVIEWER

DATE 7/18/78

RESULTS

WITH FLASHBOARDS IN PLACE, BRIDGE REMAINING INTACT -

POOL ELEVATION (FT, MSL)	SURFACE AREA (ACRES)	STORAGE VOLUME (AC-FT)	DISCHARGE (CFS)
302.4	2371	0	0
302.9	2386	1190	620
303.4	2402	2387	1900
303.9	2417	3592	3410
304.4	2433	4805	5390
305.4	2464	7254	10420
306.2	2490	9236	15470
306.9	2512	10987	18150
307.8	2540	13260	22600
308.5	2562	15046	26300

39

6000 REMOTE BATCH

M2E2FAJ

UNITED COMPUTING 67. APEX/SL D.1.5
12.52.08. 06/08/78.

M2E2FAJ

WANAQUE RESERVOIR FLOOD ROUTING

BRIDGE REMAINS IN PLACE

1) PMF WITH FLASHBOARDS

2) $\frac{1}{2}$ PMF WITH FLASHBOARDS

3) PMF WITHOUT FLASHBOARDS

4) $\frac{1}{2}$ PMF WITHOUT FLASHBOARDS

HEC-1 VERSION DATED JAN 1973
UPDATED AUG 74
CHANGE NO. 01

```
******************************
HEC-1 VERSION DATED JAN 1973
UPDATED AUG 74
CHANGE NO. 01
******************************
                         FLOOD ROUTING THROUGH WANAQUE RESERVOIR - NEW JERSEY
                                 OVERFLOW WEIR WITH FLASHBOARDS
                                PROBABLE MAXIMUM FLOOD

                                    JOB SPECIFICATION
                  NQ    NHR   NMIN   IDAY    IHR   IMIN  METRC   IPLT   IPRT  NSTAN
                  70      2     -0      1     -0     -0     -0     -0     -0     -0
                                      JOPER    NWT
                                          3     -0


          **********     **********     **********     **********     **********

                               SUB-AREA RUNOFF COMPUTATION
                  ISTAQ    ICOMP    IECON    ITAPE     JPLT     JPRT    INAME
                      1        0       -0       -0       -0       -0       -0

                                     HYDROGRAPH DATA
        IHYDG    IUHG   TAREA    SNAP   TRSDA   TRSPC   RATIO   ISNOW   ISAME   LOCAL
           -1      -0   90.40   -0.00   -0.00   -0.00  -0.000      -0      -0      -0

                                    INPUT HYDROGRAPH
       0.      100.      200.      300.      400.      500.      600.      700.      800.      900.
    1100.     1300.     1500.     1800.     2200.     2600.     3400.     4600.     7000.    10500.
   19600.    26200.    29600.    32200.    33500.    32000.    28300.    26500.    24000.    21800.
   19700.    18000.    16500.    15000.    13800.    12600.    11500.    10600.     9800.     9000.
    8300.     7700.     7100.     6600.     6200.     5800.     5400.     5000.     4700.     4400.
    4200.     4000.     3800.     3600.     3400.     3200.     3100.     3000.     2900.     2800.
    2700.     2600.     2500.     2500.     2500.     2500.     2500.     2500.     2500.     2500.

                         PEAK    6-HOUR   24-HOUR   72-HOUR   TOTAL VOLUME
                CFS     33500.   32567.    25950.    14042.       571200.
             INCHES                3.35     10.68     17.34         19.59
              AC-FT               16157.    51498.    83597.        94462.


          **********     **********     **********     **********     **********

                                  HYDROGRAPH ROUTING
                  ISTAQ    ICOMP    IECON    ITAPE     JPLT     JPRT    INAME
                      1        1       -0       -0        2       -0       -0
                                      ROUTING DATA
                           QLOSS    CLOSS      AVG     IRES    ISAME
                            -0.0   -0.000    -0.00        1       -0

                  NSTPS    NSTDL      LAG    AMSKK        X      TSK    STORA
                     -0       -0       -0   -0.000   -0.000   -0.000      -1.

 STORAGE=       0.     1190.     2387.     3592.     4805.     7254.     9236.    12247.    13260.    15046.
 OUTFLOW=       0.      620.     1800.     3410.     5390.    10420.    15470.    18150.    22600.    26300.

                         TIME   EOP STOR    AVG IN    EOP OUT
```

1	4 -0	8.	50.	4.
1	6 -0	31.	150.	16.
1	8 -0	68.	250.	35.
1	10 -0	118.	350.	61.
1	12 -0	180.	450.	94.
1	14 -0	252.	550.	131.
1	16 -0	334.	650.	174.
1	18 -0	425.	750.	222.
1	20 -0	525.	850.	273.
1	22 -0	640.	1000.	333.
2	0 -0	777.	1200.	405.
2	2 -0	935.	1400.	487.
2	4 -0	1119.	1650.	583.
2	6 -0	1338.	2000.	766.
2	8 -0	1588.	2400.	1012.
2	10 -0	1892.	3000.	1312.
2	12 -0	2303.	4000.	1717.
2	14 -0	2913.	5800.	2502.
2	16 -0	3837.	8750.	3810.
2	18 -0	5454.	15050.	6724.
2	20 -0	7724.	22900.	11617.
2	22 -0	10038.	27900.	16184.
3	0 -0	12291.	30900.	18345.
3	2 -0	14180.	32850.	24506.
3	4 -0	15343.	32750.	26916.
3	6 -0	15800.	30150.	27862.
3	8 -0	15735.	27400.	27727.
3	10 -0	15385.	25250.	27003.
3	12 -0	14806.	22900.	25803.
3	14 -0	14093.	20750.	24326.
3	16 -0	13320.	18850.	22725.
3	18 -0	12648.	17250.	19911.
3	20 -0	12115.	15750.	18033.
3	22 -0	11556.	14400.	17535.
4	0 -0	10889.	13200.	16941.
4	2 -0	10136.	12050.	16271.
4	4 -0	9332.	11050.	15555.
4	6 -0	8590.	10200.	13823.
4	8 -0	7986.	9400.	12285.
4	10 -0	7490.	8650.	11020.
4	12 -0	7071.	8000.	10044.
4	14 -0	6697.	7400.	9277.
4	16 -0	6354.	6850.	8572.
4	18 -0	6047.	6400.	7942.
4	20 -0	5773.	6000.	7378.
4	22 -0	5522.	5600.	6862.
5	0 -0	5287.	5200.	6380.
5	2 -0	5071.	4850.	5936.
5	4 -0	4875.	4550.	5534.
5	6 -0	4697.	4300.	5214.
5	8 -0	4535.	4100.	4949.
5	10 -0	4382.	3900.	4700.
5	12 -0	4237.	3700.	4462.
5	14 -0	4096.	3500.	4233.
5	16 -0	3961.	3300.	4012.
5	18 -0	3835.	3150.	3807.
5	20 -0	3725.	3050.	3627.
5	22 -0	3626.	2950.	3466.
6	0 -0	3535.	2850.	3334.
6	2 -0	3448.	2750.	3218.
6	4 -0	3364.	2650.	3105.
6	6 -0	3281.	2550.	2995.
6	8 -0	3208.	2500.	2896.
6	10 -0	3149.	2500.	2817.
6	12 -0	3101.	2500.	2754.
6	14 -0	3063.	2500.	2704.
6	16 -0	3033.	2500.	2663.

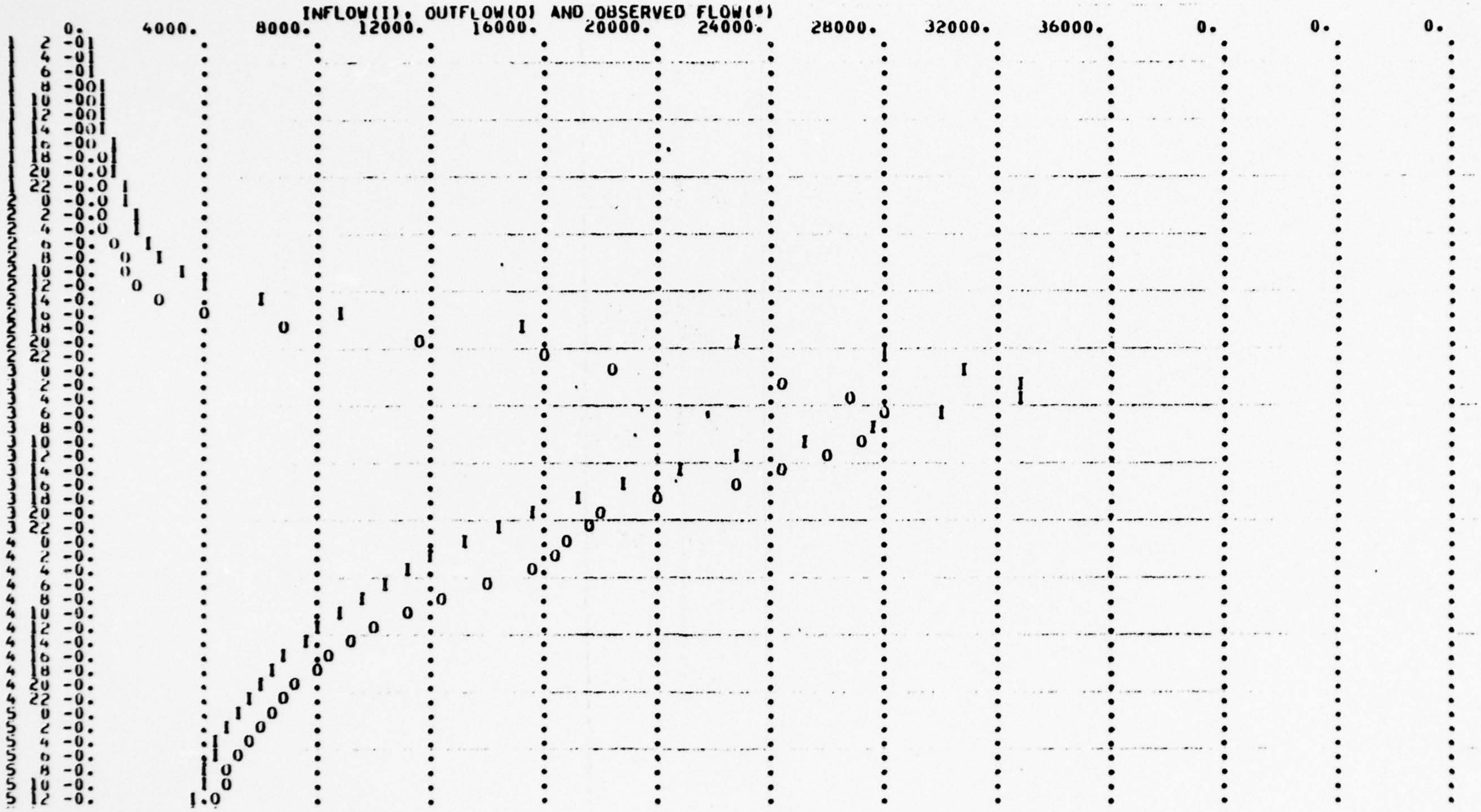
6 20 -0 2989. 2500. 2605.

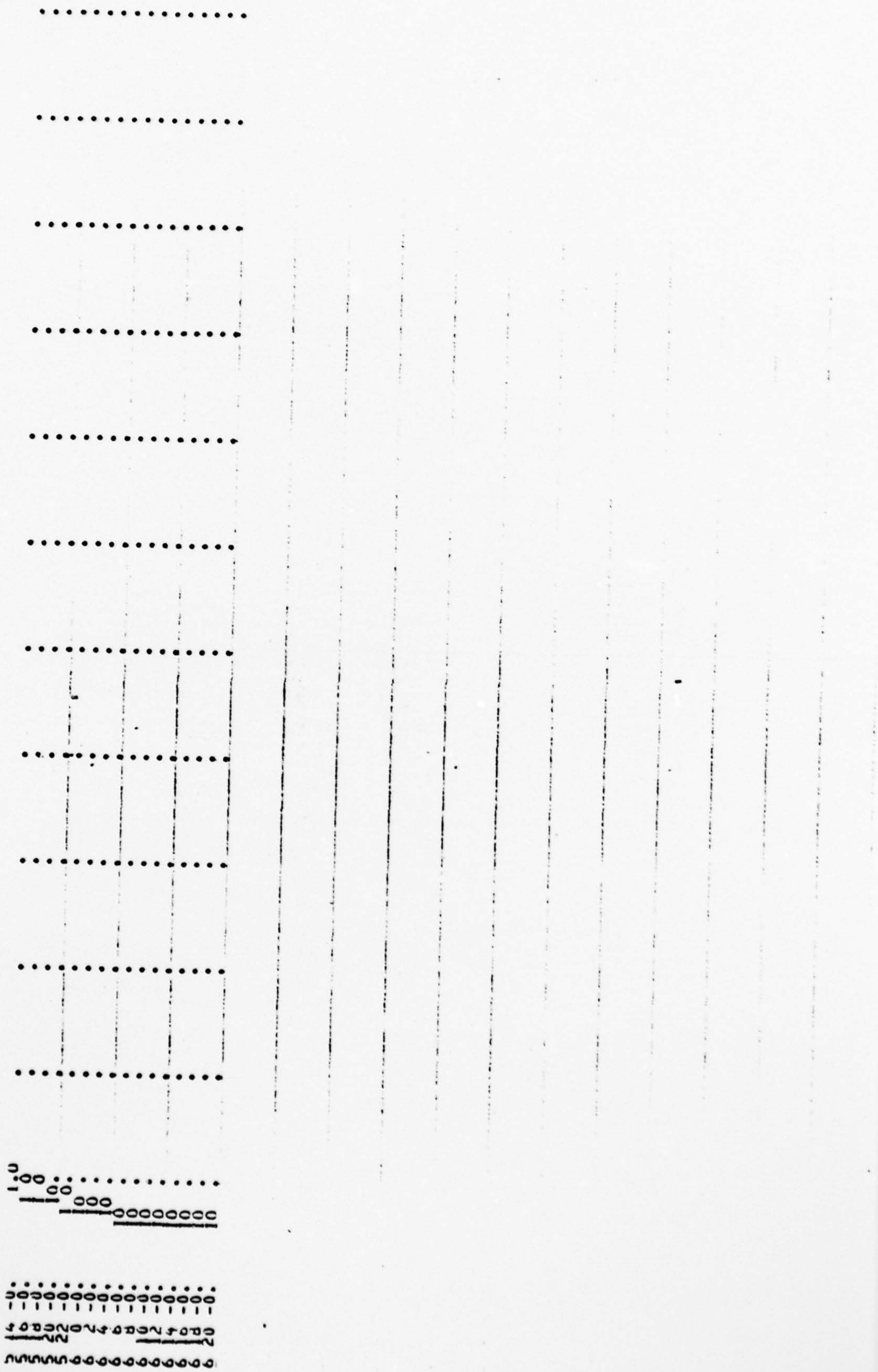
SUM 553167.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	27862.	27530.	23391.	13795.	553167.
INCHES		2.83	9.63	17.03	18.97
AC-FT		13658.	46419.	82126.	91480.

OVF

STATION 1





RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA
HYDROGRAPH AT	1	33500.	32567.	25950.	14042.	90.40
ROUTED TO	1	27862.	27530.	23391.	13795.	90.40

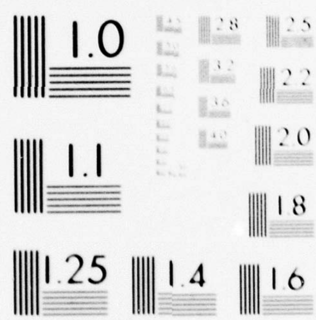
```
******************************
HEC-1 VERSION DATED JAN 1973
UPDATED AUG 74
CHANGE NO. 01
******************************
                 FLOOD ROUTING THROUGH WANAQUE RESERVOIR - NEW JERSEY
                           OVERFLOW WEIR WITH FLASHBOARDS
                     ONE-HALF THE PROBABLE MAXIMUM FLOOD

                                   JOB SPECIFICATION
                   NQ    NHR   NMIN   IDAY    IHR   IMIN METRC  IPLT  IPRT NSTAN
                   70      2     -0      1     -0     -0    -0    -0    -0    -0
                                      JOPER    NWT
                                          3     -0

    **********        **********        **********        **********        **********

                               SUB-AREA RUNOFF COMPUTATION
                   ISTAQ    ICOMP    IECON    ITAPE     JPLT     JPRT    INAME
                       1        0       -0       -0       -0       -0       -0

                                      HYDROGRAPH DATA
           IHYDG     IUHG    TAREA     SNAP    TRSDA    TRSPC    RATIO    ISNOW    ISAME    LOCAL
              -1       -0    90.40    -0.00    -0.00    -0.00   -0.000       -0       -0       -0

                                      INRUT HYDROGRAPH
        0.       50.      100.      150.      200.      250.      300.      350.      400.      450.
      550.      650.      750.      900.     1100.     1300.     1700.     2300.     3500.     5250.
     9800.    13100.    14800.    16100.    16750.    16000.    14150.    13250.    12000.    10900.
     9850.     9000.     8250.     7500.     6900.     6300.     5750.     5300.     4900.     4500.
     4150.     3850.     3550.     3300.     3100.     2900.     2700.     2500.     2350.     2200.
     2100.     2000.     1900.     1800.     1700.     1600.     1550.     1500.     1450.     1400.
     1350.     1300.     1250.     1250.     1250.     1250.     1250.     1250.     1250.     1250.

                          PEAK    6-HOUR    24-HOUR    72-HOUR   TOTAL VOLUME
                 CFS     16750.    16283.    12975.      7021.        285600.
              INCHES                 1.68      5.34       8.67           9.80
               AC-FT                8079.    25749.     41798.         47231.

    **********        **********        **********        **********        **********

                                    HYDROGRAPH ROUTING
                   ISTAQ    ICOMP    IECON    ITAPE     JPLT     JPRT    INAME
                       1        1       -0       -0        2       -0       -0
                                       ROUTING DATA
                            QLOSS    CLOSS      AVG     IRES    ISAME
                             -0.0   -0.000    -0.00        1       -0

                   NSTPS    NSTDL      LAG    AMSKK        X      TSK    STORA
                      -0       -0       -0   -0.000   -0.000   -0.000      -1.

STORAGE=      0.    1190.    2387.    3592.    4805.    7254.    9236.   12247.   13260.   15046.
OUTFLOW=      0.     620.    1800.    3410.    5390.   10420.   15470.   18150.   22600.   26300.

                            TIME   EOP STOR    AVG IN    EOP OUT
```

AD-A057 295 GILBERT ASSOCIATES INC READING PA F/G 13/2
NATIONAL DAM SAFETY PROGRAM. OVERFLOW WEIR (NJ00214), PASSAIC R--ETC(U)
JUL 78 J A HAGEN DACW61-78-C-0114

UNCLASSIFIED NL

2 OF 2
AD
A057 295

END
DATE
FILMED
9-78
DDC



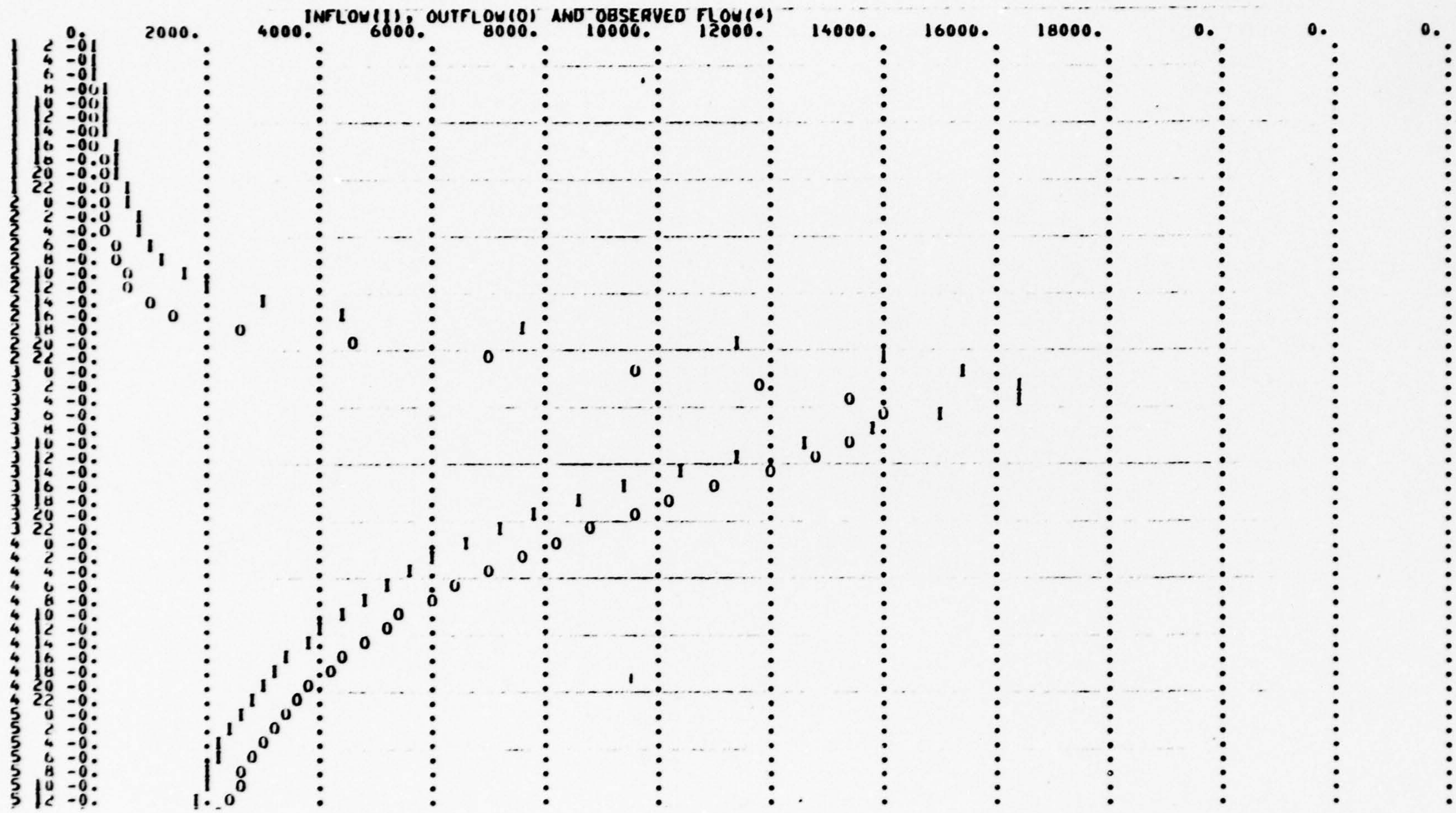
MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

1	4	-0	4.	25.	2.
1	6	-0	16.	75.	8.
1	8	-0	34.	125.	18.
1	10	-0	59.	175.	31.
1	12	-0	90.	225.	47.
1	14	-0	126.	275.	66.
1	16	-0	167.	325.	87.
1	18	-0	213.	375.	111.
1	20	-0	262.	425.	137.
1	22	-0	320.	500.	167.
2	0	-0	389.	600.	203.
2	2	-0	468.	700.	244.
2	4	-0	560.	825.	292.
2	6	-0	672.	1000.	350.
2	8	-0	807.	1200.	420.
2	10	-0	978.	1500.	509.
2	12	-0	1213.	2000.	643.
2	14	-0	1558.	2900.	983.
2	16	-0	2077.	4375.	1494.
2	18	-0	2982.	7525.	2595.
2	20	-0	4285.	11450.	4541.
2	22	-0	5630.	13950.	7085.
3	0	-0	6812.	15450.	9512.
3	2	-0	7771.	16425.	11737.
3	4	-0	8404.	16375.	13350.
3	6	-0	8640.	15075.	13950.
3	8	-0	8605.	13700.	13863.
3	10	-0	8436.	12625.	13433.
3	12	-0	8166.	11450.	12743.
3	14	-0	7842.	10375.	11919.
3	16	-0	7502.	9425.	11051.
3	18	-0	7168.	8625.	10243.
3	20	-0	6833.	7875.	9555.
3	22	-0	6500.	7200.	8872.
4	0	-0	6179.	6600.	8212.
4	2	-0	5870.	6025.	7578.
4	4	-0	5580.	5525.	6982.
4	6	-0	5314.	5100.	6436.
4	8	-0	5069.	4700.	5932.
4	10	-0	4842.	4325.	5466.
4	12	-0	4629.	4000.	5104.
4	14	-0	4425.	3700.	4770.
4	16	-0	4229.	3425.	4450.
4	18	-0	4047.	3200.	4153.
4	20	-0	3879.	3000.	3879.
4	22	-0	3722.	2800.	3622.
5	0	-0	3573.	2600.	3384.
5	2	-0	3430.	2425.	3194.
5	4	-0	3293.	2275.	3011.
5	6	-0	3165.	2150.	2840.
5	8	-0	3048.	2050.	2683.
5	10	-0	2939.	1950.	2537.
5	12	-0	2836.	1850.	2400.
5	14	-0	2739.	1750.	2271.
5	16	-0	2647.	1650.	2147.
5	18	-0	2562.	1575.	2034.
5	20	-0	2486.	1525.	1932.
5	22	-0	2418.	1475.	1841.
6	0	-0	2355.	1425.	1769.
6	2	-0	2295.	1375.	1709.
6	4	-0	2236.	1325.	1651.
6	6	-0	2179.	1275.	1595.
6	8	-0	2126.	1250.	1543.
6	10	-0	2081.	1250.	1499.
6	12	-0	2043.	1250.	1461.
6	14	-0	2011.	1250.	1429.
6	16	-0	1984.	1250.	1402.

```
    6 18 -0      1960.      1250.      1379.
    6 20 -0      1941.      1250.      1360.

           SUM                       273915.

             PEAK    6-HOUR   24-HOUR   72-HOUR   TOTAL VOLUME
   CFS      13950.   13749.   11686.     6819.       273915.
INCHES                1.41      4.81      8.42          9.40
 AC-FT               6821.    23190.    40599.        45298.
```

OVF

STATION 1



RUNOFF SUMMARY, AVERAGE FLOW

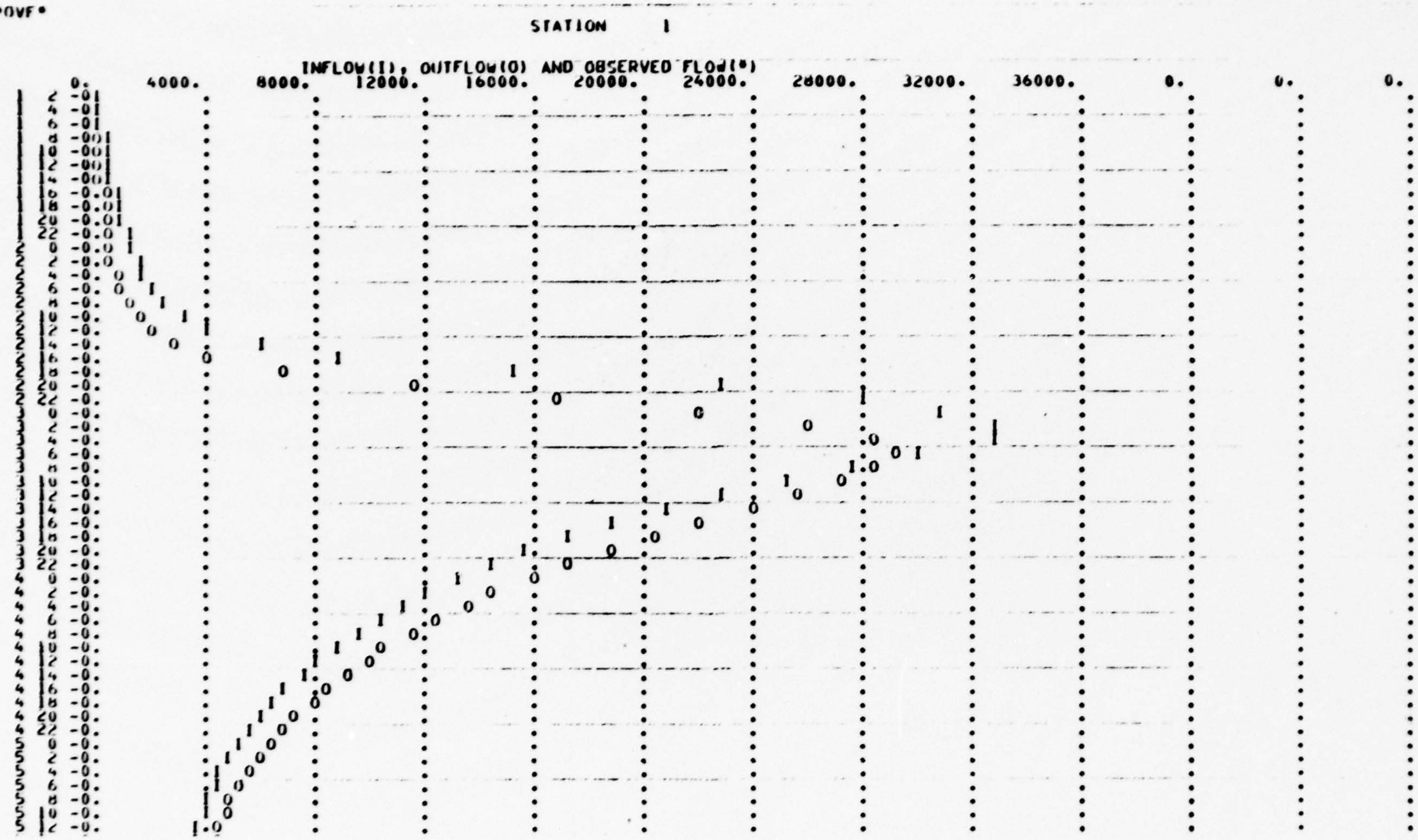
		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA
HYDROGRAPH AT	1	16750.	16283.	12975.	7021.	90.40
ROUTED TO	1	13950.	13749.	11686.	6819.	90.40

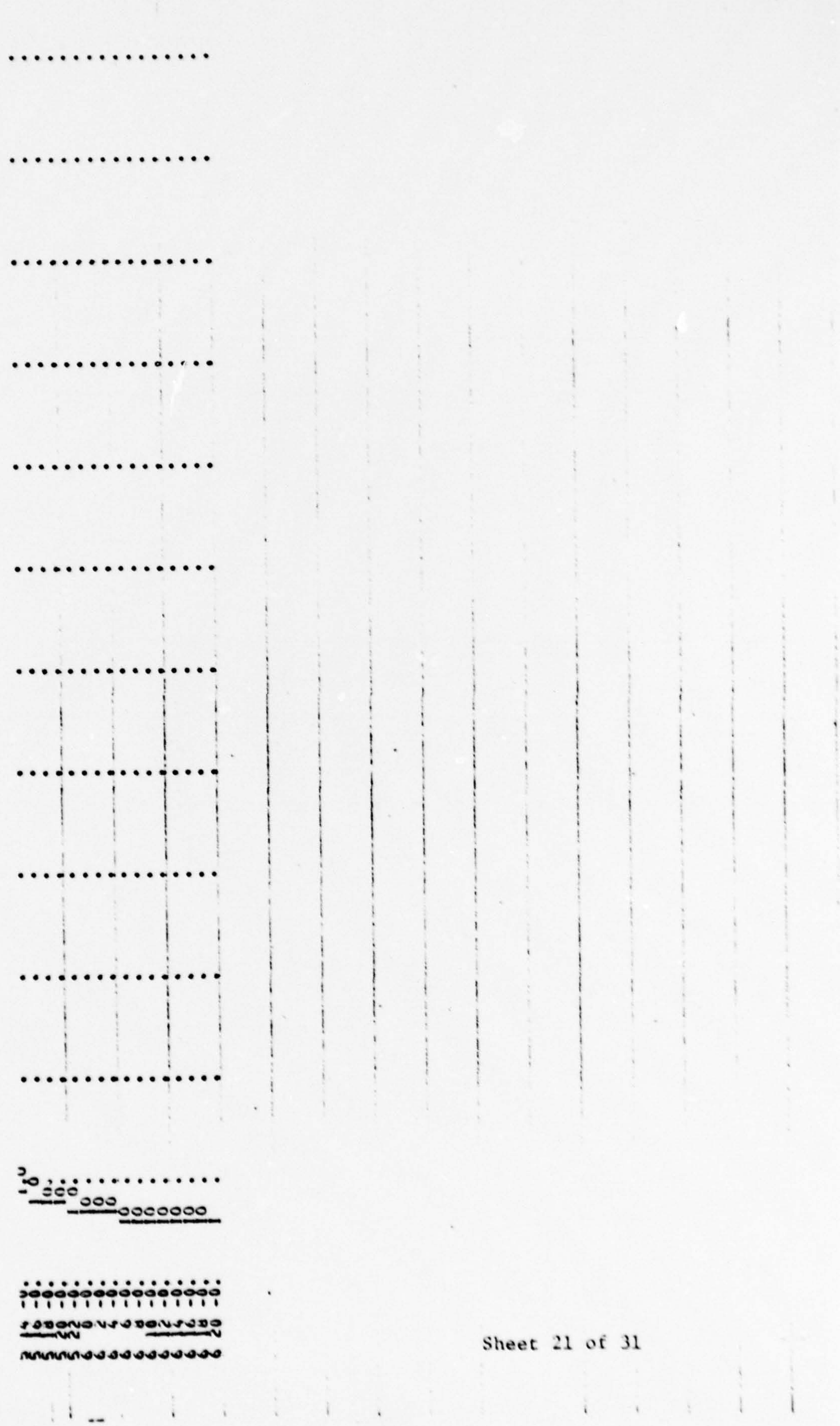
```
******************************
HEC-1 VERSION DATED JAN 1973
UPDATED AUG 74
CHANGE NO. 01
******************************
                         FLOOD ROUTING THROUGH WANAQUE RESERVOIR - NEW JERSEY
                              OVERFLOW WEIR WITHOUT FLASHBOARDS
                                  PROBABLE MAXIMUM FLOOD

                                      JOB SPECIFICATION
               NQ     NHR    NMIN    IDAY    IHR   IMIN METRC   IPLT   IPRT NSTAN
               70       2      -0       1     -0     -0    -0     -0     -0    -0
                                      JOPER    NWT
                                          3     -0

   **********      **********      **********      **********      **********

                              SUB-AREA RUNOFF COMPUTATION
               ISTAQ    ICOMP    IECON    ITAPE    JPLT     JPRT    INAME
                   1        0       -0       -0      -0       -0       -0

                                     HYDROGRAPH DATA
   IHYDG    IUHG   TAREA     SNAP    TRSDA    TRSPC    RATIO    ISNOW    ISAME    LOCAL
      -1      -0   90.40    -0.00    -0.00    -0.00   -0.000       -0       -0       -0

                                    INPUT HYDROGRAPH
      0.      100.     200.     300.     400.     500.     600.     700.     800.     900.
   1100.     1300.    1500.    1800.    2200.    2600.    3400.    4600.    7000.   10500.
  19600.    26200.   29600.   32200.   33500.   32000.   28300.   26500.   24000.   21800.
  19700.    18000.   16500.   15000.   13800.   12600.   11500.   10600.    9800.    9000.
   8300.     7700.    7100.    6600.    6200.    5800.    5400.    5000.    4700.    4400.
   4200.     4000.    3800.    3600.    3400.    3200.    3100.    3000.    2900.    2800.
   2700.     2600.    2500.    2500.    2500.    2500.    2500.    2500.    2500.    2500.

                       PEAK   6-HOUR   24-HOUR   72-HOUR   TOTAL VOLUME
            CFS      33500.   32567.    25950.    14042.        571200.
         INCHES                 3.35     10.68     17.34          19.59
          AC-FT               16157.    51498.    83597.         94462.

   **********      **********      **********      **********      **********

                                   HYDROGRAPH ROUTING
               ISTAQ    ICOMP    IECON    ITAPE    JPLT     JPRT    INAME
                   1        1       -0       -0       2       -0       -0
                                     ROUTING DATA
                        QLOSS    CLOSS      AVG    IRES    ISAME
                         -0.0   -0.000    -0.00       1       -0

               NSTPS    NSTDL      LAG    AMSKK       X      TSK    STORA
                  -0       -0       -0   -0.000  -0.000   -0.000      -1.

 STORAGE=       0.    1160.    2320.    3490.    4670.    7060.    9470.   11920.   13650.   14150.
 OUTFLOW=       0.     725.    2050.    3750.    5760.   10550.   16180.   22520.   27290.   28680.

                         TIME   EOP STOR    AVG IN    EOP OUT
```

1	2 -0	0.	0.	0.
1	4 -0	8.	50.	5.
1	6 -0	31.	150.	19.
1	8 -0	67.	250.	42.
1	10 -0	115.	350.	72.
1	12 -0	175.	450.	104.
1	14 -0	244.	550.	153.
1	16 -0	322.	650.	201.
1	18 -0	408.	750.	255.
1	20 -0	502.	850.	314.
1	22 -0	610.	1000.	381.
2	0 -0	738.	1200.	462.
2	2 -0	886.	1400.	554.
2	4 -0	1058.	1650.	661.
2	6 -0	1204.	2000.	844.
2	8 -0	1499.	2400.	1113.
2	10 -0	1784.	3000.	1436.
2	12 -0	2171.	4000.	1880.
2	14 -0	2753.	5800.	2679.
2	16 -0	3646.	8750.	4016.
2	18 -0	5233.	15050.	6888.
2	20 -0	7493.	22400.	11562.
2	22 -0	9752.	27900.	16909.
3	0 -0	11657.	30900.	21839.
3	2 -0	13142.	32850.	25889.
3	4 -0	14065.	32750.	28444.
3	6 -0	14294.	30150.	29081.
3	8 -0	14068.	27400.	28453.
3	10 -0	13638.	25250.	27256.
3	12 -0	13051.	22900.	25639.
3	14 -0	12393.	20750.	23825.
3	16 -0	11721.	18850.	22006.
3	18 -0	11074.	17250.	20330.
3	20 -0	10450.	15750.	18716.
3	22 -0	9862.	14400.	17195.
4	0 -0	9316.	13200.	15819.
4	2 -0	8793.	12050.	14599.
4	4 -0	8302.	11050.	13451.
4	6 -0	7851.	10200.	12399.
4	8 -0	7436.	9400.	11426.
4	10 -0	7051.	8650.	10532.
4	12 -0	6692.	8000.	9812.
4	14 -0	6350.	7400.	9127.
4	16 -0	6027.	6850.	8480.
4	18 -0	5732.	6400.	7889.
4	20 -0	5464.	6000.	7352.
4	22 -0	5216.	5600.	6854.
5	0 -0	4981.	5200.	6384.
5	2 -0	4764.	4850.	5948.
5	4 -0	4563.	4550.	5578.
5	6 -0	4378.	4300.	5263.
5	8 -0	4210.	4100.	4976.
5	10 -0	4054.	3900.	4710.
5	12 -0	3907.	3700.	4461.
5	14 -0	3768.	3500.	4224.
5	16 -0	3634.	3300.	3996.
5	18 -0	3512.	3150.	3787.
5	20 -0	3403.	3050.	3624.
5	22 -0	3304.	2950.	3480.
6	0 -0	3211.	2850.	3345.
6	2 -0	3123.	2750.	3217.
6	4 -0	3040.	2650.	3096.
6	6 -0	2959.	2550.	2979.
6	8 -0	2888.	2500.	2876.
6	10 -0	2833.	2500.	2795.
6	12 -0	2789.	2500.	2732.
6	14 -0	2755.	2500.	2682.
6	16 -0	2728.	2500.	2643.

```
6 18 -0       2707.      2500.      2612.
6 20 -0       2691.      2500.      2588.

        SUM                         554967.

           PEAK     6-HOUR    24-HOUR    72-HOUR   TOTAL VOLUME
   CFS    29081.    28659.    24056.     13815.        554967.
INCHES                2.95      9.90      17.06          19.04
 AC-FT               14219.    47739.     82246.         91777.
```

OVF


STATION 1
INFLOW(I), OUTFLOW(O) AND OBSERVED FLOW(*)
0.
4000.
8000.
12000.
16000.
20000.
24000.
28000.
32000.
36000.
0.
0.
0.



Sheet 21 of 31

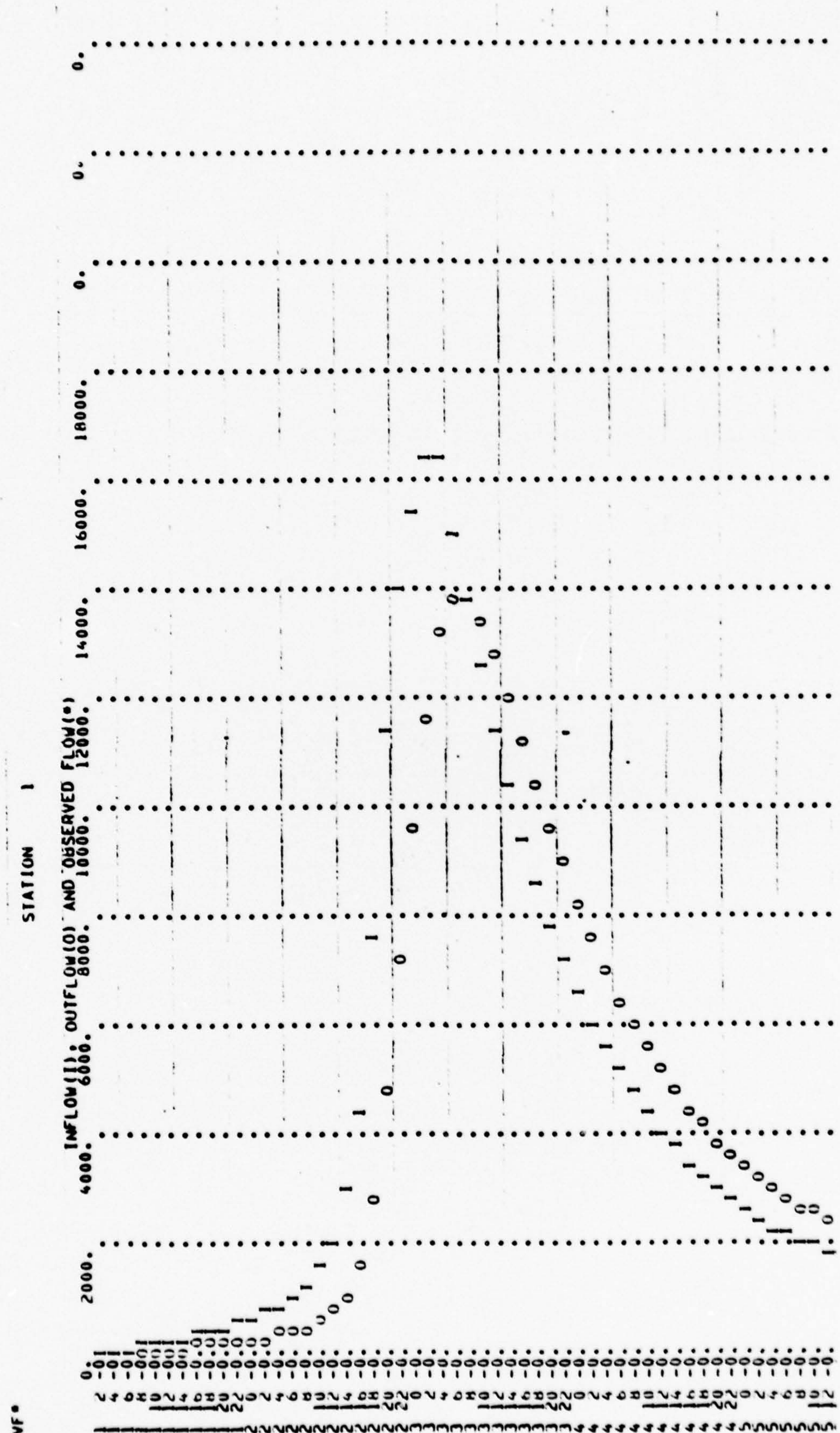
RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA
HYDROGRAPH AT	1	33500.	32567.	25950.	14042.	90.40
ROUTED TO	1	29081.	28659.	24056.	13815.	90.40

```
******************************
HEC-1 VERSION DATED JAN 1973
UPDATED AUG 74
CHANGE NO. 01
******************************
              FLOOD ROUTING THROUGH WANAQUE RESERVOIR - NEW JERSEY
                        OVERFLOW WEIR WITHOUT FLASHBOARDS
             ONE-HALF THE PROBABLE MAXIMUM FLOOD

                              JOB SPECIFICATION
               NQ    NHR   NMIN   IDAY    IHR   IMIN METRC   IPLT   IPRT NSTAN
               70      2     -0      1     -0     -0    -0     -0     -0    -0
                                  JUPER         NWT
                                      3          -0

     **********      **********      **********      **********      **********

                              SUB-AREA RUNOFF COMPUTATION
               ISTAQ    ICOMP    IECON    ITAPE     JPLT     JPRT    INAME
                   1        0       -0       -0       -0       -0       -0
                                   HYDROGRAPH DATA
        IHYDG     IUHG    TAREA     SNAP    TRSDA    TRSPC    RATIO    ISNOW    ISAME    LOCAL
           -1       -0    90.40    -0.00    -0.00    -0.00   -0.000       -0       -0       -0
                                   INPUT HYDROGRAPH
       0.      50.     100.     150.     200.     250.     300.     350.     400.     450.
     550.     650.     750.     900.    1100.    1300.    1700.    2300.    3500.    5250.
    9800.   13100.   14800.   16100.   16750.   16000.   14150.   13250.   12000.   10900.
    9850.    9000.    8250.    7500.    6900.    6300.    5750.    5300.    4900.    4500.
    4150.    3850.    3550.    3300.    3100.    2900.    2700.    2500.    2350.    2200.
    2100.    2000.    1900.    1800.    1700.    1600.    1550.    1500.    1450.    1400.
    1350.    1300.    1250.    1250.    1250.    1250.    1250.    1250.    1250.    1250.

                      PEAK    6-HOUR   24-HOUR   72-HOUR   TOTAL VOLUME
              CFS    16750.    16283.    12975.     7021.       285600.
           INCHES                1.68      5.34      8.67          9.80
            AC-FT               8079.    25749.    41798.        47231.

     **********      **********      **********      **********      **********

                                 HYDROGRAPH ROUTING
               ISTAQ    ICOMP    IECON    ITAPE     JPLT     JPRT    INAME
                   1        1       -0       -0        2       -0       -0
                                   ROUTING DATA
                        QLOSS    CLOSS      AVG     IRES    ISAME
                         -0.0   -0.000    -0.00        1       -0

               NSTPS    NSTDL      LAG    AMSKK        X      TSK    STORA
                  -0       -0       -0   -0.000   -0.000   -0.000      -1.

STORAGE=        0.    1160.    2320.    3490.    4670.    7060.    9470.   11920.   13650.   14150.
OUTFLOW=        0.     725.    2050.    3750.    5760.   10550.   16180.   22520.   27290.   28680.

                    TIME  EOP STOR    AVG IN   EOP OUT
```

1	2	-0	0.	0.	0.
1	4	-0	4.	25.	2.
1	6	-0	15.	75.	10.
1	8	-0	33.	125.	21.
1	10	-0	58.	175.	36.
1	12	-0	87.	225.	55.
1	14	-0	122.	275.	76.
1	16	-0	161.	325.	101.
1	18	-0	204.	375.	128.
1	20	-0	251.	425.	157.
1	22	-0	305.	500.	191.
2	0	-0	369.	600.	231.
2	2	-0	443.	700.	277.
2	4	-0	529.	825.	331.
2	6	-0	634.	1000.	396.
2	8	-0	761.	1200.	475.
2	10	-0	922.	1500.	576.
2	12	-0	1145.	2000.	710.
2	14	-0	1476.	2900.	1086.
2	16	-0	1973.	4375.	1653.
2	18	-0	2847.	7525.	2816.
2	20	-0	4110.	11450.	4806.
2	22	-0	5418.	13950.	7260.
3	0	-0	6580.	15450.	9588.
3	2	-0	7538.	16425.	11667.
3	4	-0	8190.	16375.	13191.
3	6	-0	8451.	15075.	13801.
3	8	-0	8438.	13700.	13768.
3	10	-0	8279.	12625.	13398.
3	12	-0	8009.	11450.	12768.
3	14	-0	7678.	10375.	11993.
3	16	-0	7322.	9425.	11162.
3	18	-0	6968.	8625.	10366.
3	20	-0	6615.	7875.	9658.
3	22	-0	6267.	7200.	8960.
4	0	-0	5932.	6600.	8289.
4	2	-0	5611.	6025.	7646.
4	4	-0	5310.	5525.	7043.
4	6	-0	5035.	5100.	6491.
4	8	-0	4781.	4700.	5982.
4	10	-0	4543.	4325.	5544.
4	12	-0	4319.	4000.	5163.
4	14	-0	4107.	3700.	4802.
4	16	-0	3908.	3425.	4462.
4	18	-0	3725.	3200.	4150.
4	20	-0	3558.	3000.	3867.
4	22	-0	3402.	2800.	3623.
5	0	-0	3251.	2600.	3403.
5	2	-0	3107.	2425.	3194.
5	4	-0	2971.	2275.	2997.
5	6	-0	2847.	2150.	2815.
5	8	-0	2734.	2050.	2651.
5	10	-0	2630.	1950.	2501.
5	12	-0	2534.	1850.	2361.
5	14	-0	2444.	1750.	2230.
5	16	-0	2358.	1650.	2106.
5	18	-0	2279.	1575.	2003.
5	20	-0	2207.	1525.	1921.
5	22	-0	2140.	1475.	1844.
6	0	-0	2076.	1425.	1772.
6	2	-0	2016.	1375.	1703.
6	4	-0	1959.	1325.	1638.
6	6	-0	1904.	1275.	1575.
6	8	-0	1855.	1250.	1519.
6	10	-0	1815.	1250.	1473.
6	12	-0	1781.	1250.	1434.
6	14	-0	1753.	1250.	1403.
6	16	-0	1730.	1250.	1376.

```
6 18 -0       1711.      1250.      1354.
6 20 -0       1695.      1250.      1336.

      SUM                          275387.

            PEAK    6-HOUR   24-HOUR   72-HOUR   TOTAL VOLUME
   CFS     13801.   13656.    11693.     6848.       275387.
INCHES                1.41      4.81      8.46          9.45
 AC-FT                6775.    23205.    40768.        45542.
```


STATION 1
INFLOW(I), OUTFLOW(O) AND OBSERVED FLOW(*)
0.
2000.
4000.
6000.
8000.
10000.
12000.
14000.
16000.
18000.
0.
0.
0.
OVF

RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA
HYDROGRAPH AT	1	16750.	16283.	12975.	7021.	90.40
ROUTED TO	1	13801.	13656.	11693.	6848.	90.40

```
M2E2FAJ.   06/08/78. *UNITED COMPUTING* 67. APEX/SL D.1.5

12.49.12$RAOUL,CM100000,T20,  RAP 6547 067249000
12.49.12.0674 4695 UCS
12.49.13$AC3400G32A303,067249000 .B
12.49.13. 06/08/78,M2E2FAJ
12.49.13$FL    32768      0.002        5         3.
12.49.13$FI        0          1
12.49.13.GET,MECI(LIBRARY)
12.49.14.READY -    MECI
12.49.14.RFL,150000.
12.49.14$FL     4096      0.002        0         0.
12.49.14$FI        0          1
12.49.27.MECI.
12.49.27$FLS   53248      0.002
12.49.29$FLR   53248      0.190      349        18
12.49.29.FL REQUIRED TO LOAD      141322B ( 49874)
12.49.29.FL REQUIRED TO EXECUTE   123700B ( 42944)
12.49.29$FL    53248      0.202
12.49.35.STOP
12.49.35.GOEXIT.
12.49.35. ILLEGAL CONTROL CARD.
12.49.35.EXIT.
12.49.35$SB*   42944      1.326      773        66
12.49.35.COST.
12.49.35$FL    42944      1.327
12.49.35.             SERV. UNITS-2 =     12
12.49.35.              JOB COST=$       3.00
12.49.35$JT03  12288      1.345        7         5
12.49.35.    *FL*   *CPU SEC. *DISC PRUS* DISC ACC
12.49.35.* LDR PRUS*P.F. ACC *TAPE PRUS* TAPE ACC
```

APPENDIX E

INSPECTION REPORTS

(The North Jersey District Water Supply Commission provided the inspection reports contained herein.)

NORTH JERSEY DISTRICT WATER SUPPLY COMMISSION

M E M O R A N D U M

TO: Dam Inspection File

FROM: Joseph Foley, Engineer

DATE: April 5, 1977

On March 31, 1977 Roscoe Jennings, Doug De Lorie and I inspected the dams at the Wanaque Reservoir; the following is a report on their conditions and recommendations on maintenance of same.

FURNACE ROAD DAM

Condition: There are trees and brush on the wet and dry sides of the dam and also a small swamp of apparently trapped water behind the dam.

Recommendations: The trees should be killed and removed using poison suitable for potable water.

MIDVALE DAM

Condition: Some trees are growing on the wet and dry sides of the dam. There is a small spring flowing from the foot of the dam at the north end. Wet spots and soft wet sand are also apparent at the foot of the dam. No sink holes or other indications of dam failure were apparent at this location. A sample of water from this spring and a sample from the reservoir were taken and anlyzed, the results are as follows:

Spring Water:	Specific conductivity	68
	pH	6.3
Reservoir Water:	Specific conductivity	102
	pH	6.9

The results indicate that this water is more likely to be ground water than reservoir water. (For additional information, please refer to a memo from Bob Wieland to George Destito dated May 3, 1975).

Recommendations: The trees on the dam should be killed and removed. The dam should also be checked periodically to be sure the spring is not a leak in the dam.

RAYMOND DAM

Condition: Excellent

SPILLWAY (Overflow Weir)

Condition: Good, except that it was indicated by Ernie Restaino that there is a small leak in the spillway. I did not observe it because of the overflow. I will check it again when the reservoir goes down.

Recommendations: The leak in the spillway should be fixed when the reservoir goes down.

WOLF DEN DAM

Condition: There are trees and shrubs on both the wet and dry sides. There are small springs flowing from the low sections behind the dam. Some samples were also taken here and the results were that the water had a specific conductivity of 90 and a pH of 6.3, so this water is most likely ground water also.

Recommendations: I recommend that the trees and shrubs be removed.

GREEN SWAMP

#4 Dam

Condition: The general condition of the dam is good, although sections of the gunite surfacing are cracked and have fallen off (especially near the expansion joints), due to moisture that found its way under the guite. There was water running out of the drain but this flow was not excessive.

Recommendations: The cracked and loose gunite should be chipped away and replaced and at the expansion joints, the gunite should be chipped and tar poured in to allow expansion of the concrete.

#3 and #2A Dams

Condition: Both small dams are heavily wooded and there is a small swamp behind the #3 dam.

Recommendations: The only recommendation for these dams is that the trees be removed from both sides of the dams.

#2 Dam

Condition: This dam is in excellent condition, except around the expansion joints where the gunite is cracked due to the fact that no allowance was made for expansion when the guite was applied to the dam. There is also a swamp behind this dam, but this looks like a natural swamp.

Recommendations: The gunite at the expansion joints should be chipped away and tar poured in to allow expansion and any other cracks in the gunite should be chipped and repaired.

#1 Dam

Condition: There are trees and shrubs on both wet and dry sides of this dam. There is also a swamp behind the dam.

Recommendations: The dam should be cleared of trees and shrubs.

As a result of my research, so far on dam inspection, I received a booklet, "Supervision of Dams by State Authorities" published by the United States Committee on large dams, July 1966. This publication had little information on the actual inspection of dams but it did have some useful information such as: the function of dam supervision in New Jersey is performed by the Chief Engineer, Division of Water Policy and Supply, Department of Conservation and Economic development. Inspection of dams is done by the State at the State's own expense on the complaint of potential failure.

Additional information on dam inspection is also coming from the Corps of Engineers and the United States Committee on Large Dams.

JF:lk

cc: Dean C. Noll
Robert G. Wieland

Report on Dam Inspection

WANAQUE PROJECT

Application No. 32.

Location 23.31.5.4.9 and nearby.

On March 23, 1928, the gates in the main dam were closed except for the passage of 27 m. g. d. through the blow-off, and on March 29, 1928, the water in the reservoir had risen 7 feet.

On March 29, 1928, in company with Mr. H. T. Critchlow, inspection was made of all of the dams in the Wanaque project.

Furnace Road dam was found to be about 50 per cent complete.

Post Brook Diversion dam, weir and control house were complete except for closing a small breach which was left in the dam for stream control, and installation of recording gage in the control house.

Wanaque Main dam.

Midvale Dam.

Overflow Weir.

Wolf Den Dam, and

Green Swamp Dams Nos. 1, 2, 3 and 4 were complete and were given final inspection.

The construction of all dams has been done in accordance with the approved plans and in a thoroughly workmanlike and satisfactory manner.

John N. Brooks

John N. Brooks
Hydraulic Engineer.

C 7/30

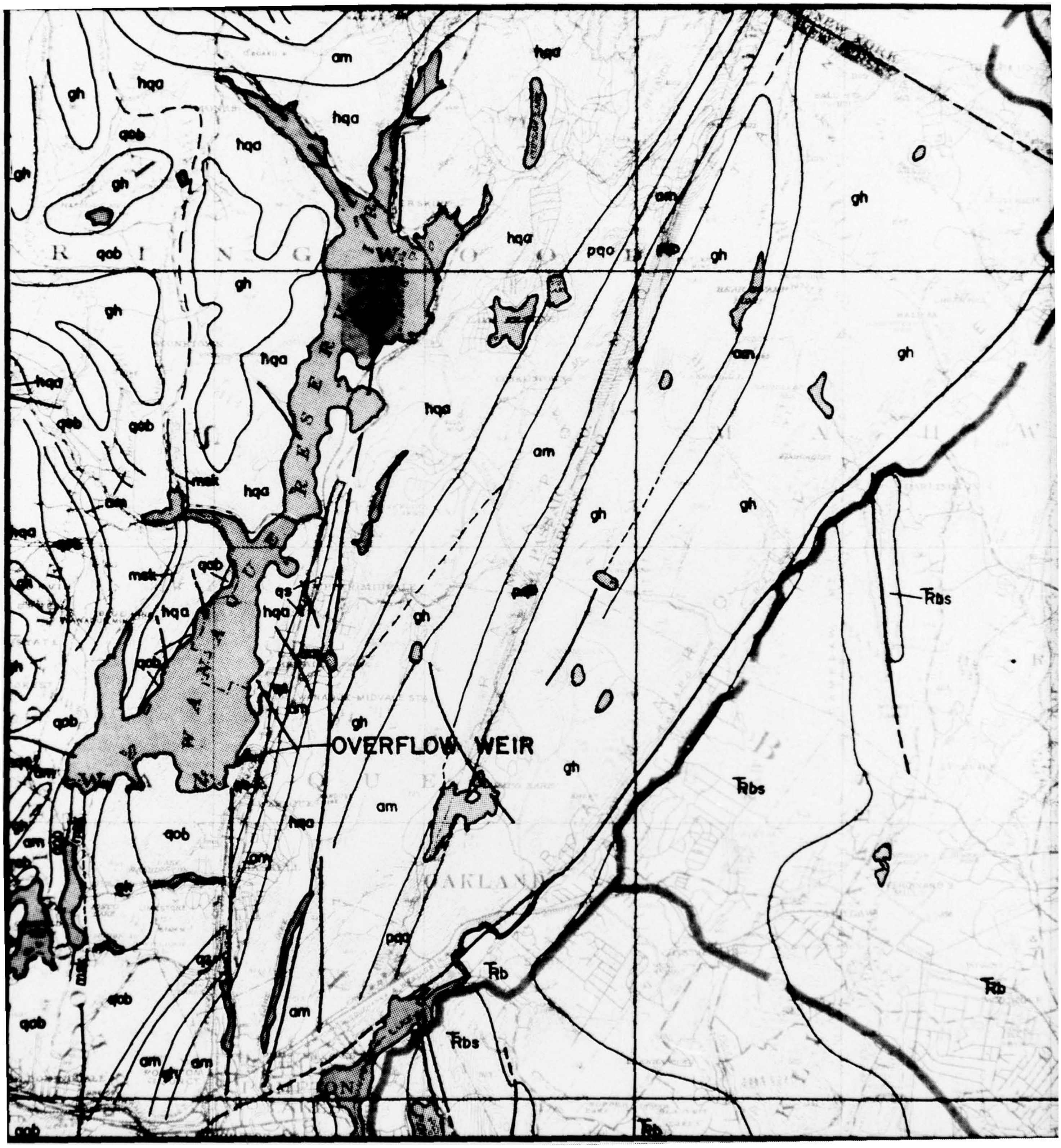
Trenton, N. J.

March 30, 1928.

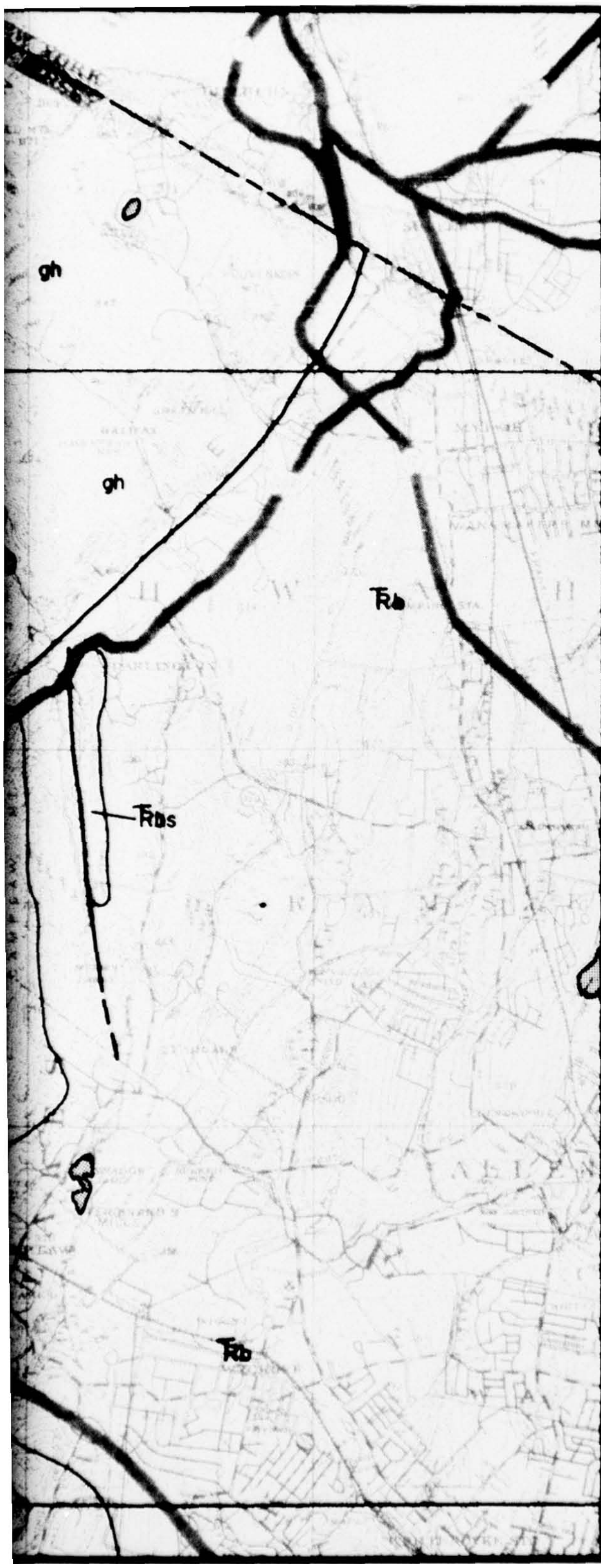
(New Jersey - Dept. of Environmental Protection)

APPENDIX F

GEOLOGIC MAPS



OVERFLOW WEIR
OAKLAND
am
hqa
gh
qab
pqa
mak
qs
T̶Rbs
T̶Rb



LEGEND

TRIASSIC

TRb	BRUNSWICK FORMATION
TRbs	BASALT FLOWS

PRECAMBRIAN

gh	MOSTLY HORNBLENDE GRANITE AND GRANITE GNEISS
am	AMPHIBOLITE
pqo	PYROXENE GNEISS; MAINLY QUARTZ-OLIGOCLASE - CLINOPYROXENE GNEISS
hqa	PYROXENE GNEISS; MAINLY QUARTZ-ANDESINE GNEISS WITH BOTH ORTHO-AND CLINOPYROXENE
qo	QUARTZ-OLIGOCLASE-GNEISS
qob	QUARTZ-OLIGOCLASE-BIOTITE GNEISS
qs	SILLIMANITE GNEISS
msk	MARBLE AND SKARN

CONTACT LINE
FAULT LINE

NOTES:

1. THE PRECAMBRIAN MAP UNITS REPRESENT GENERALIZED GROUPINGS OF ROCK TYPES BASED MAINLY ON MINERAL COMPOSITION. THERE IS MUCH LOCAL VARIATION IN THE MINERAL COMPOSITION.
2. THE CONTACT LINES AND FAULT LINE SHOWN ON THE DRAWING ARE DASHED WHERE INFERRED.

SOURCE:

NEW JERSEY GEOLOGICAL SURVEY TOPOGRAPHIC SERIES AND GEOLOGIC OVERLAY SHEETS 23.

1 0 1 2
SCALE IN MILES

APPENDIX F
REGIONAL GEOLOGIC MAP
SHOWING DAM LOCATION

APPENDIX G

REFERENCES

APPENDIX G

REFERENCES

1. Recommended Guidelines for Safety Inspection of Dams, (Washington, D.C., Department of the Army, Office of the Chief of Engineers).

2. Public Works, Vol. 54, No. 5, May 1923.

3. North Jersey District Water Supply Commission - Report 1925, (Newark, N.J., Office of the Commission), 1925.

4. Engineering and Design - Gravity Dam Design Stability, (Washington, D.C., Department of the Army, Office of the Chief of Engineers), 1974.

5. Design of Small Dams - 2nd Ed., (Washington, D.C., Department of the Interior), 1973.

6. Daily Reservoir Water Level and Discharge Record Files from October 1950 to date by the NJDWSC.

7. Water Resources Data for New Jersey, Part 1, Surface Water Records, USGS, Department of the Interior.

8. "Passaic River Basin - New Jersey and New York Survey Report for Water Resources," New York District Corps of Engineers, June 1972.

9. HEC-1 Flood Hydrograph Package, Hydrologic Engineering Center, Corps of Engineers, January 1973.

APPENDIX H

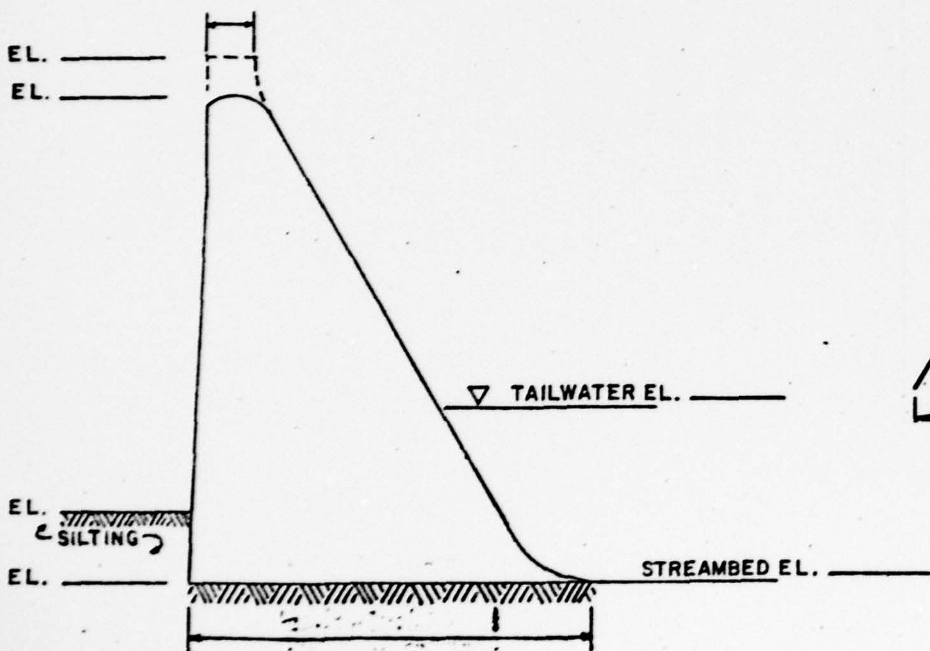
STABILITY ANALYSIS

GRAVITY DAM DESIGN
STABILITY ANALYSIS

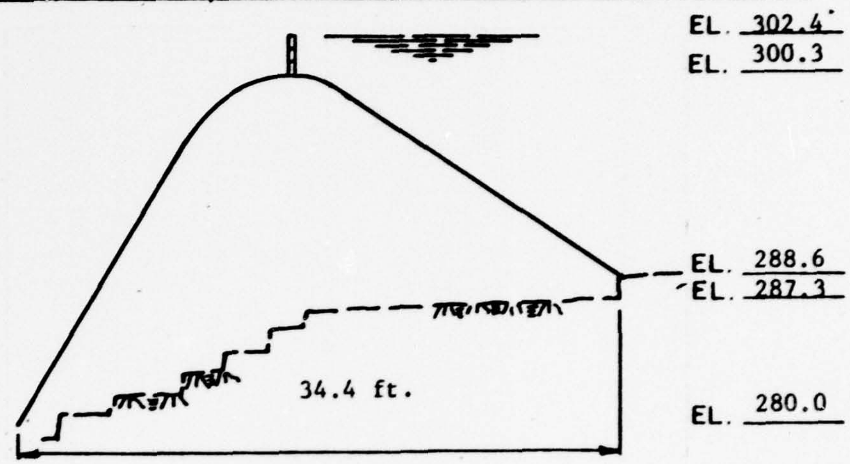
Wanaque Reservoir
Overflow Weir

ANALYSIS DONE ON ___ FULL SECTION X PARTIAL SECTION
LOCATION OF SECTION Max. Height of Dam
ANALYSIS PREPARED BY R. F. Noll/J. A. Hagen 6/28/78

LOADING CASE	ELEV. HEAD WATER	ELEV. TAIL WATER	ΣV	ΣH	$\frac{\Sigma H}{\Sigma V}$	LOCATION RESULTANT FROM TOE	% BASE IN COMPRESSION	FACTOR SAFETY SLIDING	FOUNDATION PRESSURE TOE	FOUNDATION PRESSURE HEEL
PMF	308.6	290.0	18.1 K	9.9 K	0.55	6.24'	54%	333	1.93 K/s.f.	0
1/2 PMF	306.0	287.0	24.1	9.0	0.37	10.1'	88%	483	1.59	0
1' of Ice @ Top StopLogs	302.4	280.0	35.6	12.1	0.34	10.1'	88%	358	2.35	0



FULL SECTION



PARTIAL SECTION

APPENDIX I

AUXILIARY DAM

APPENDIX I

Summary of Auxilliary Dam G-G

1. A small saddle dam is located approximately 100-ft south of the Overflow Weir at the rim of Wanaque Reservoir. No formal inspection on this dam was made because it was not within the scope of this contract agreement. Because this dam is shown on the record drawing for the Overflow Weir and was casually and briefly visited by us during the field inspection trip, we wish to present some essential data for this dam and to discuss some visual findings related to the safety of the dam in this Appendix. For location of this dam and typical sections, see Figure 3 of the main report.

2. Data on this dam are tabulated below:

   Type: Earthfill with concrete core wall.
   Length: 56 ft measured along the crest.
   Height: 16.5 ft (structural), 8.5 ft (hydraulic).
   Top Elevation: 310.0
   Top width: 15 ft.
   Side slopes: U/S 2 (H): 1 (V) with thick riprap lining.
   D/S 2 (H): 1 (V) with grass cover.
   Impervious core: Concrete core wall.
   Cutoff: concrete core wall to final rock surface.
   Grout curtain: None.

3. The dam crest and upstream face appears to be in good condition as defined in Appendix J, hence the downstream embankment slope and toe area were not inspected in the field. The riprap slope on the upstream face is rather uniform and apparently stable. Excessive tree growth was observed on the embankment. Removal of trees and tree stumps is necessary in the near future in order to prevent further spread and deep penetration of root systems.

4. On the basis of the record drawing, the foundation of this dam appears to be sound and stable, as the enlarged concrete base of the core wall is founded on the probably sound gneissic rocks and the presence of weak foundation soils is unlikely.

5. Although the dam will not be overtopped by the probable maximum flood (PMF), the top of the concrete core wall will probably be exceeded by the PMF. Excessive seepage and a dangerous piping condition at the toe area are likely to occur when the reservoir water percolates through the unknown embankment material. Further investigation and analysis on the structural stability and seepage need to be done the PMF load condition in the future.

APPENDIX J

CONDITIONS

APPENDIX J

CONDITIONS

This report is based on a visual inspection of the dam, a review of available engineering data and a hydrologic analysis performed during Phase I Investigation as set forth in the Recommended Guidelines for Safety Inspection of Dams, as modified by the contract between the U.S. Corps of Engineers and Gilbert Associates, Inc., Contract No. DACW61-78-C-0114.

The foregoing review, inspection, and analysis are by their nature limited in scope. It is possible that hazardous conditions exist and that conditions exist which with time might develop into safety hazards and that these conditions are not detectable by means of the aforesaid review, inspection, and analysis. Accordingly Gilbert Associates, Inc. cannot and does not warrant or represent that conditions which are hazardous do not exist, or that conditions do not exist which with time might develop into safety hazards.

As required by the Corps of Engineers the terms "good", "fair", "poor", "condition" have been used in this Report to characterize the information obtained from the aforesaid review, inspection, and analysis. The definitions of these terms as used are:

"good condition" - minor studies or remedial measures are required.

"fair condition" - sizeable studies or remedial measures are required due to deficiencies which could be hazardous depending on conditions. Immediate attention is required.

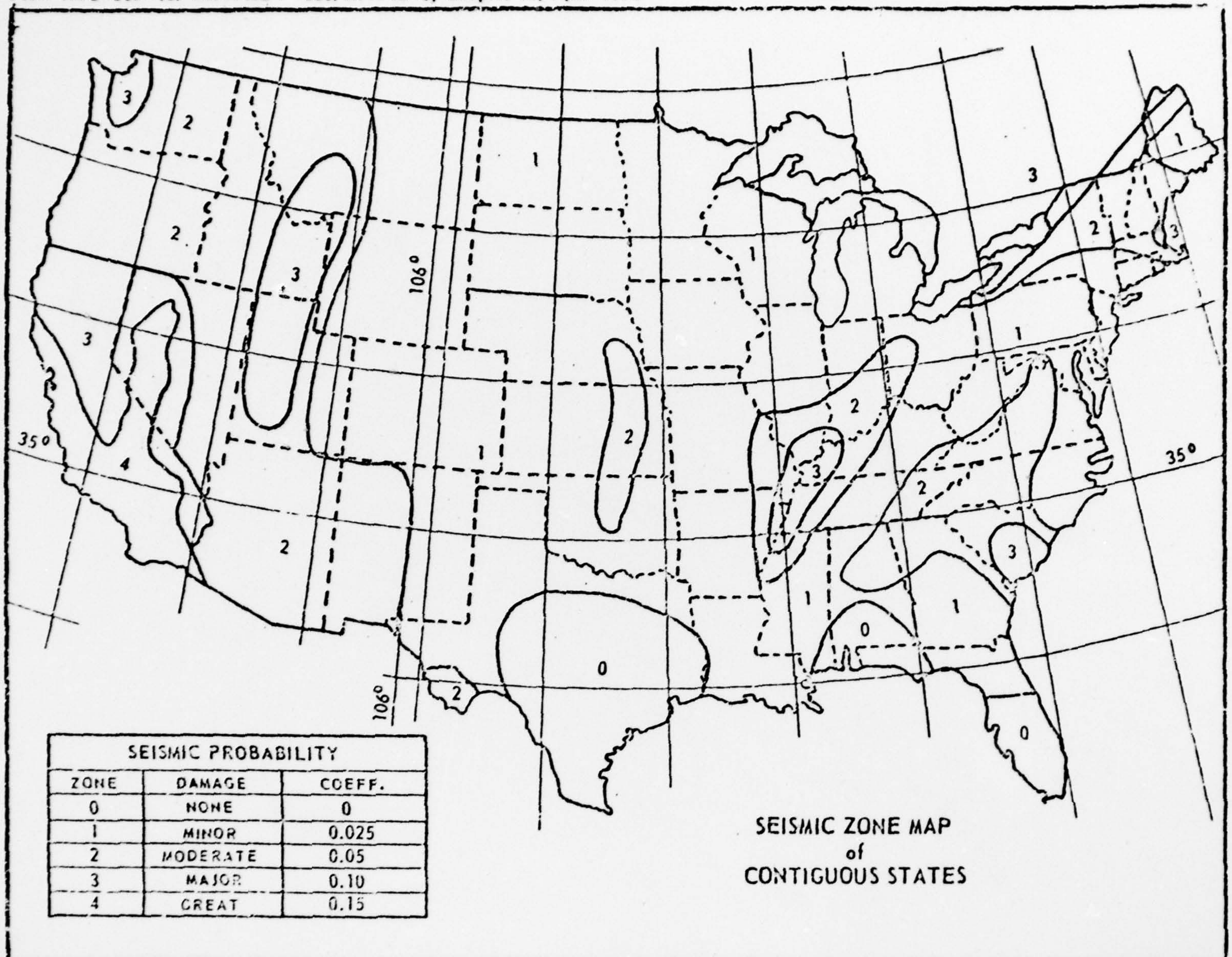
"poor condition" - major studies or remedial measures are required due to deficiencies which could be hazardous depending on conditions. Immediate studies or corrective action is required.

APPENDIX K

CORPS OF ENGINEERS GUIDELINE STUDIES

4.4.2.1. Seismic Stability. The inertial forces for use in the conventional equivalent static force method of analysis should be obtained by multiplying the weight by the seismic coefficient and should be applied as a horizontal force at the center of gravity of the section or element. The seismic coefficients suggested for use with such analyses are listed in Figures 1 through 4. Seismic stability investigations for all high hazard category dams located in Seismic Zone 4 and high hazard dams of the hydraulic fill type in Zone 3 should include suitable dynamic procedures and analyses. Dynamic analyses for other dams and higher seismic coefficients are appropriate if in the judgment of the investigating engineer they are warranted because of proximity to active faults or other reasons. Seismic stability investigations should utilize "state-of-the-art" procedures involving seismological and geological studies to establish earthquake parameters for use in dynamic stability analyses and, where appropriate, the dynamic testing of materials. Stability analyses may be based upon either time-history or response spectra techniques. The results of dynamic analyses should be assessed on the basis of whether or not the dam would have sufficient residual integrity to retain the reservoir during and after the greatest or most adverse earthquake which might occur near the project location.

From TM 5-809-10/NAVFAC P-355/AFM 88-3, Chapter 13; April 1973



SEISMIC PROBABILITY		
ZONE	DAMAGE	COEFF.
0	NONE	0
1	MINOR	0.025
2	MODERATE	0.05
3	MAJOR	0.10
4	GREAT	0.15

Figure 1